THE CONTENTS OF THIS SECTION ARE
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INITIAL gj DATE 9/27/02

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Appendix A

Stress Induced in Geomembrane and Geosynthetic Clay Liner

STRESS INDUCED IN GEOMEMBRANE AND GCL

OBJECTIVE: Estimate the stress induced in the geomembrane and geocomposite clay (GCL) liner due to the self and overlying material weight and subgrade settlement

METHOD: The stress induced in the geomembrane and GCL are due a combination of the weight of the material and the strain applied to the material due to settlement of the underlying layers.

ASSUMPTIONS: Use GSE 60 mil textured HDPE geomembrane and Cetco GCL

CALCULATION:

Self and overlying weight: Based on EDF-ER-268, the minimum interface shear strength for the liner system is 29.3°. The liner system slope angle is 3H:1V (18.4°); therefore, since the interface friction angle is greater than the slope angle, there is no net stress on the liner system.

Settlement:

Minimum horizontal slope length:	85 ft	See page 2, Drawing C-302
Minimum vertical slope length:	28 ft	See page 2, Drawing C-302
Initial three dimensional slope length (l_o):	89.49 ft	
Maximum vertical displacement	1.2 ft	Based on results of EDF-ER-266
Final vertical slope length:	29.2 ft	
Final three dimensional length (l_f):	89.88 ft	
Liner strain $((l_f - l_o)/l_o)$:	0.004258	
	0.425783 %	

HDPE Liner Stress (s = Ev)

HDPE Tensile Strength at Yield:	130 lb/in	See specification sheet
HDPE liner thickness:	0.060 in	
HDPE liner elastic modulus:	2,166.667 psi	
Liner stress (s):	9.225299 psi	
Safety Factor (Allowable elastic stress/applied stress):	234.86	

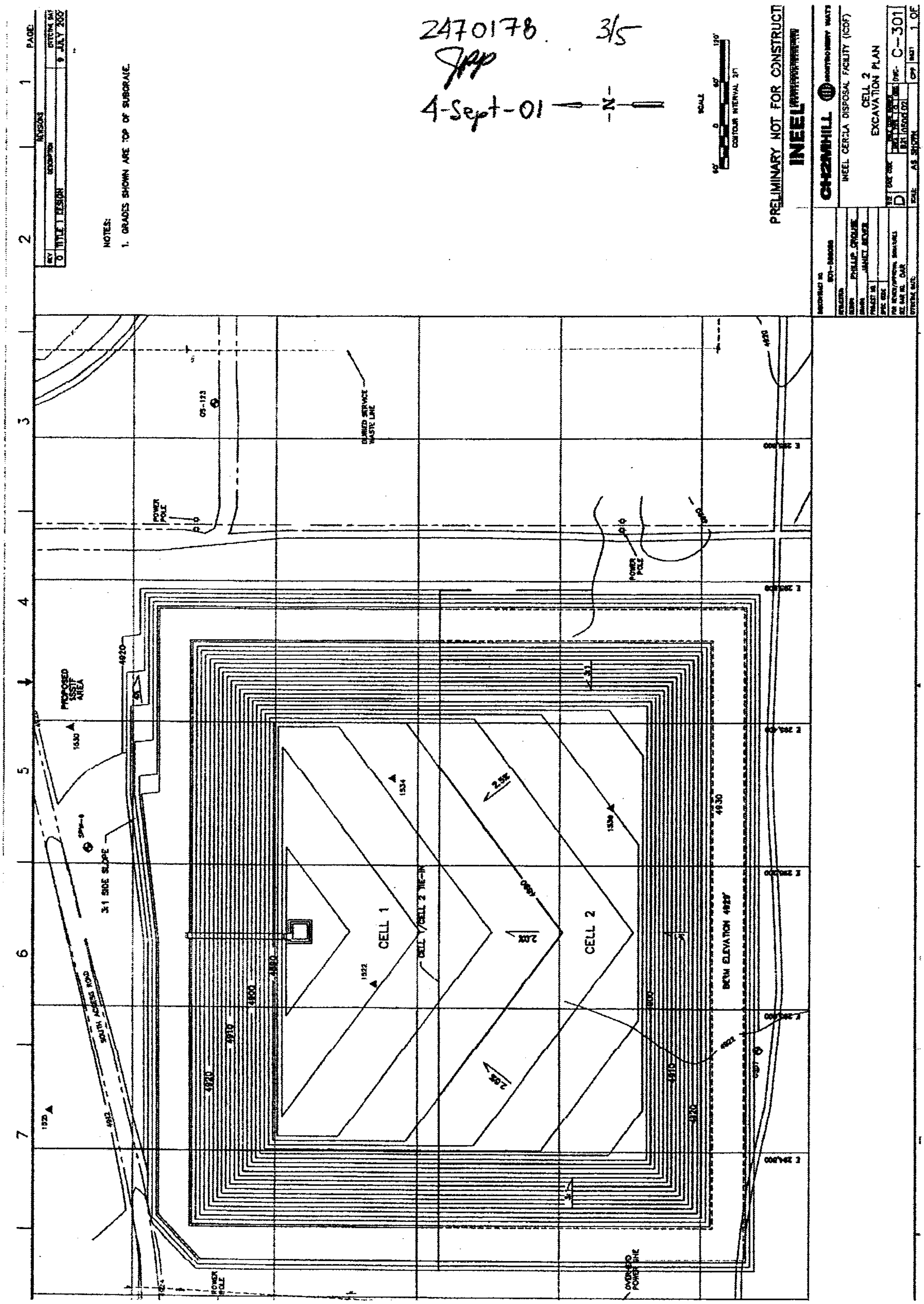
CONCLUSIONS: Based on the calculations, the subgrade settlement will have no detrimental effect on the HDPE liner system.

GCL Liner Stress

Elastic properties for GCL material are not normally determined. A standard physical property specified is grab elongation. Therefore the estimated strain will be compared to the GCL grab elongation.

Estimated strain from above calculation:	0.425783 %	
GCL grab elongation:	50 %	per company representative
Safety Factor (Allowable strain/applied strain)	117.43	

CONCLUSIONS: Based on the calculations, the settlement of the subgrade will have no detrimental effect on the GCL liner system.



(See Drawing C-301 in the Draft Final Drawings for more detail.)

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Solution: (a) Using a maximum strain of 33%, the value of $f(\epsilon)$ required grab tensile strength is as follows:

$$
\begin{aligned}
T_{reqd} &= p'(d_v)^2(0.52) \\
&= p'(0.33\ d_a)^2(0.52) \\
&= 0.057\ p'd_a^2 \\
&= 0.057(100)(2.0)^2 \\
&= 22.6\ \text{lb.}
\end{aligned}
$$

(b) The global factor of safety on a 125-lb. ultimate grab tensile geotextile with partial factors of safety of 2.5 is as follows:

$$
\begin{aligned}
FS &= \frac{T_{ult}}{T_{reqd}} \\
&= \frac{125/2.5}{22.6} \\
&= 2.2, \text{ which is acceptable.}
\end{aligned}
$$

2.5.4 Puncture Resistance

Although not only related to the separation function, the geotextile during its placement must survive the installation process. Indeed, fabric survivability is critical in all types of applications; without it, the best of designs are futile (recall Section 2.2.5.1). In this regard, sharp stones, tree stumps, roots, miscellaneous debris, and other things on the ground beneath the geotextile could puncture through the geotextile after stone base and traffic loads are imposed above it. The design method suggested for this situation is shown schematically in Figure 2.29. For these conditions, the vertical force exerted on the geotextile (which is gradually tightening around the protruding object) is as follows:

$$F_{reqd} = p'd_a^2 S_1 S_2 S_3 \tag{2.30}$$

where F_{reqd} = the required vertical force to be resisted,
p' = the pressure exerted on the geotextile (approximately 100% of tire inflation pressure at the ground surface for small stone thicknesses),
d_a = the average diameter of the puncturing aggregate or sharp object,
S_1 = protrusion factor = h_h/d_a,
h_h = protrusion height $\leq d_a$,
S_2 = scale factor to adjust ASTM D4833 test value using 5/16-in.-diameter puncture probe to actual puncturing object = $0.31/d_a$,
S_3 = shape factor to adjust flat puncture probe of ASTM D4833 to actual shape of puncturing object = $1 - A_p/A_c$ (values of A_p/A_c to be used

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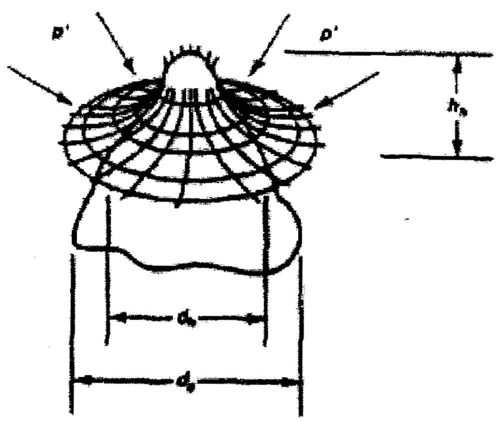


Figure 2.28 Visualization of a stone puncturing a geotextile as pressure is applied from above.

range from 0.8 for Ottawa sand, $\boxed{\text{0.7 for run-of-bank gravel}}$, 0.4 for crushed rock, and 0.3 for shot rock),
A_p = projected area of particle, and
A_c = area of smallest circumscribed circle.

Example:

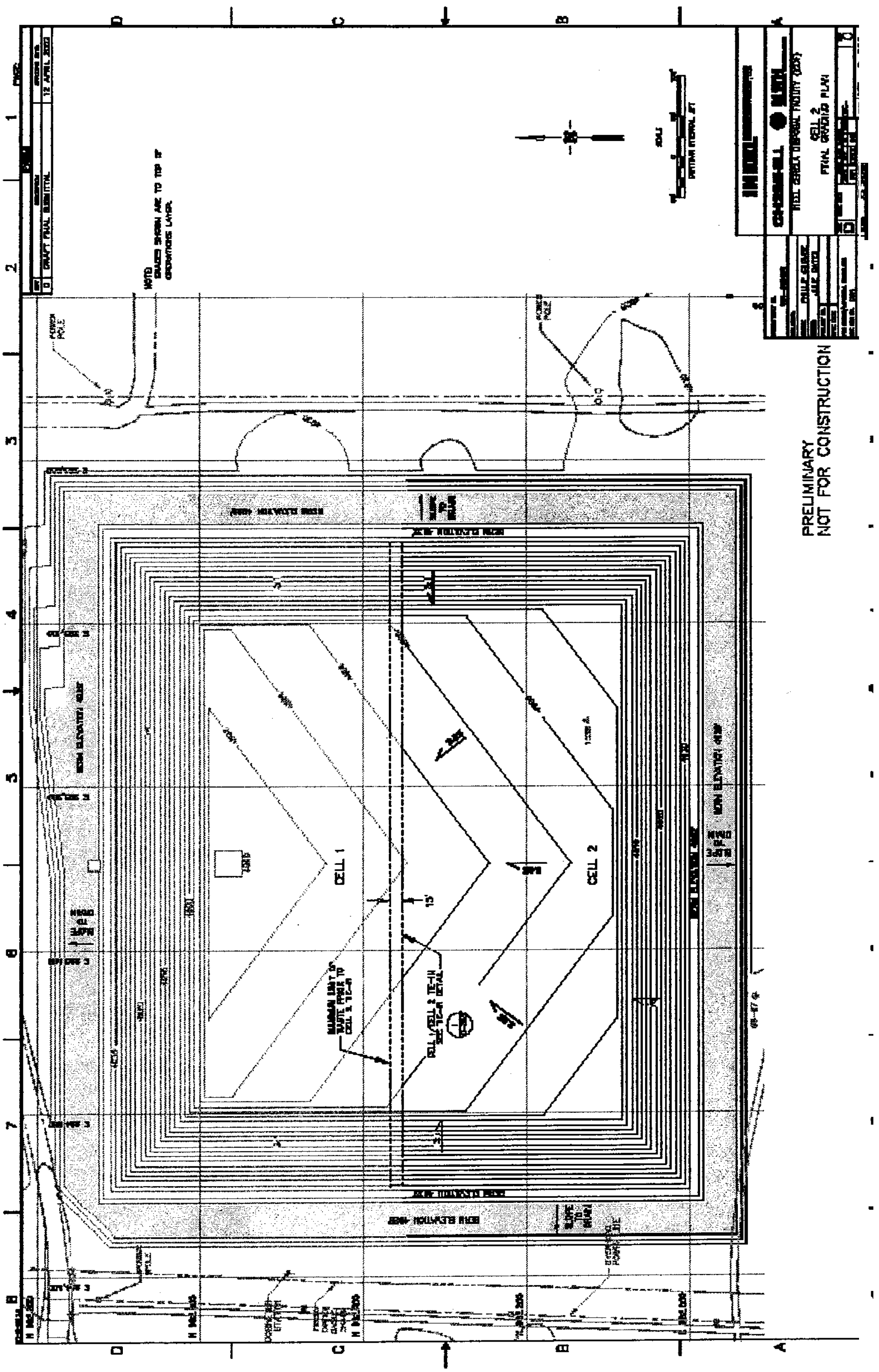
What is the factor of safety against puncture of a geotextile from a 2.0-in. stone by a loaded truck with tire inflation pressure of 80 lb./in.² traveling on the surface of the stone base? The geotextile has an ultimate puncture strength of 45 lb. according to ASTM D4833.

Solution: *Using the full stress on the geotextile of 80 lb./in.² and factors of* 0.33, 0.155, and 0.6 for S_1, S_2, and S_3, respectively,

$$F_{reqd} = p' d_a^2 S_1 S_2 S_3$$
$$= 80 \times (2.0)^2 (0.33)(0.155)(0.6)$$
$$= 9.82 \text{ lb.}$$

Assuming that the cumulative partial factor of safety is 2.0, the global factor of safety is as follows:

$$FS = \frac{F_{ult}}{F_{reqd}}$$
$$= \frac{45/2.0}{9.82}$$
$$= 2.3, \text{ which is acceptable.}$$



(See Drawing C-302 of the ICDF Draft Final Drawings for details)

ENGINEERING DESIGN FILE

Functional File No. NA
EDF No. ER-268

1. Project File No.: NA 2. Project/Task: ICDF

3. Subtask: Slope Stability Assessments

4. Title: Slope Stability Assessments (Title I)

5. Summary:

This report documents the slope stability evaluations that were performed to aid in the design of the liner system for the ICDF landfill and ICDF evaporation pond. These stability evaluations included veneer stability, global stability, and stability after excavation. Veneer stability involves evaluation of the potential for sliding of the drainage layer on the liner system before refuse is placed. Global stability involves evaluation of the potential for sliding during operation of the landfill and of the stability of the final landfill configuration with the cover in place. Stability after excavation involves evaluation of stability, immediately after excavation of the landfill and before placement of the lining system.

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D. Vernon, MS 3930
T. Borschel, MS 3930
Distribution (summary package only):

7. Review (R) and Approval (A) Signatures:
(Minimum reviews and approvals are listed. Additional reviews/approvals may be added.)

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Independent Reviewer	R	Don Anderson/ CH2M HILL		07/09/01
Requestor	A	Mike Reimbold/ CH2M HILL		07/09/01
Approver	A	Jay Dehner/CH2M HILL		07/09/01

For veneer stability of the lining system, strength values based on test data conducted under low normal stresses were considered appropriate. For this project, low normal stress—in the context of veneer stability—was limited to stress levels less than 600 pounds per square foot (psf), or an equivalent of up to about 5 ft of soil. Interface shear strength data applicable to this stress level were then modeled using linear regression. In the regression analysis, the interface shear strength was represented by an effective friction angle by forcing the cohesion intercept to zero. The idea of using the effective friction angle to represent the shear strength of the interface at low normal stress is to maintain the magnitude of the shear strength while eliminating the dependency on the cohesion intercept in the strength parameter determination. For low effective confining pressures, this approach allows the shear strength to approach zero as confinement goes to zero.

Appendix A contains the database of interface shear strength tests that were analyzed. Material interfaces in which test data has been analyzed under this task include soil/geocomposite, textured HDPE/geocomposite, textured HDPE/GCL, and GCL/geocomposite interface. Based on the measured and reported interface strength data, peak and residual strengths of lining material interfaces were evaluated. For veneer stability analysis, however, residual strengths are considered to be appropriate (Stark and Poeppel 1994). For the soil/geocomposite interface shear strength, test data that indicate a mixture of sand and gravel (with and without silt) for the soil component, consistent with the description of the on-site native material, were evaluated in the analyses. Shear strength data for CCL/textured HDPE interface were not analyzed due to the inadequate amount of data that is available. In the absence of adequate data, test results from the Cedar Hills Regional Landfill (CHRL) project (CH2M HILL 1998a) were used in the analyses. These results indicate an interface friction angle of about 25 degrees and a cohesion of zero for the CCL/HDPE interface. Additional site-specific testing is recommended to confirm this value, as discussed in "Evaluation of Geotechnical Investigations and Calculations Required to Complete Design and Construction" (DOE-ID 2001b). Results of site-specific interface shear testing will be reported in the 90% Remedial Design/Remedial Action (RD/RA) design submittal. Analyses presented herein will be revised if lower strength values are obtained from the site-specific testing.

Based on the above evaluations, the critical interface for the veneer stability analysis appears to be the non-woven GCL/non-woven geocomposite interface. A residual friction angle of 19 degrees was developed from the existing data for low normal stress for this interface. Most recent test results provided by Montgomery Watson (1999) using exactly the same materials proposed for this project, except that the woven side of GCL was used, indicate an effective residual interface friction angle of 24 degrees. In this project, it is proposed that a non-woven side of GCL will be placed in contact with the geocomposite, which, as a result, could yield a higher residual friction angle than the 24 degrees that was reported. For this reason, and the fact that actual test results are available for the proposed lining material, it was decided to use a residual friction angle of 24 degrees for the GCL/geocomposite interface. This value matches the residual interface friction angle for the HDPE/geocomposite interface as the most critical interface for veneer stability. It is recommended, however, that actual interface shear strength tests be conducted for the non-woven GCL/non-woven geocomposite interface to confirm this value. Results of site-specific interface shear testing will be reported in the 90% RD/RA design submittal. Analyses presented herein will be revised if lower strength values are obtained from the site-specific testing.

The analysis for self-weight (Case 1) involved an evaluation of veneer stability under the load of the 3-ft-thick operations layer only. For equipment loads (Case 2), an equivalent equipment weight of 4,400 pounds per lineal foot of lining system such as that caused by a D6H Caterpillar dozer was assumed during placement of the drainage layer over the HDPE geomembrane. It was further assumed for this loading case that the seepage height would be zero. For the seepage case (Case 3), the maximum allowable head over the side slope lining system is 6 in. for stability purposes. FSs corresponding to seepage heights of 3 in. and 6 in. were evaluated. A maximum slope height of 40 ft was used in running the SLOPBASE program.

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ENGINEERING DESIGN FILE

Functional File No. NA
EDF No. EDF-ER-266

1. Project File No.: NA 2. Project/Task: ICDF

3. Subtask: Subsurface Consolidation Calculation

4. Title: Subsurface Consolidation Calculation

5. Summary:
The purpose of this calculation is to determine the maximum consolidation in the landfill foundation soils resulting from the load of the waste material and cover. The maximum amount of consolidation is used to determine the differential settlement and integrity of the landfill liner system.

6. Distribution (complete package):
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T. Borschel, MS 3930
Distribution (summary package only):

7. Review (R) and Approval (A) Signatures:
(Minimum reviews and approvals are listed. Additional reviews/approvals may be added.)

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Independent Reviewer	R	Phillip Crouse/Montgomery Watson		
Requestor	A			
Approver	A	Jay Dehner/CH2M HILL		

6. RESULTS AND CONCLUSIONS

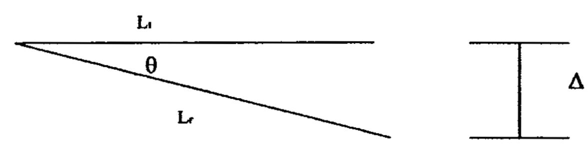
6.1 Maximum Differential Settlement

The maximum total settlement at the center of the landfill is conservatively estimated to be 1.2 ft. Differential settlement is a function of the maximum total settlement and will be less than the total settlement; however, it is difficult to estimate. So, as a worst case, the maximum differential settlement is assumed to be equal to the maximum total settlement of 1.2 ft.

6.2 Stress and Strain in Liner Components

As the bottom of the liner consolidates, it will distort creating strain in each of the liner components. Assuming all the settlement occurs near the center of the landfill and no settlement occurs on the ends, the maximum differential settlement will be 1.2 ft as described previously. The floor of the landfill in its shortest direction is approximately 528 ft. (EDF 265 – Air Space Volume Calculation. The resulting strain is calculated below:

$$\varepsilon = \frac{L_f - L_I}{L_I}$$



Where,

ε = Strain

L_f = Final length

L_I = The length on which the distortion acts

θ = Angle of rotation

Δ = Distortion

Using half of the width of the landfill, the maximum amount of strain is 0.001%. The calculation is presented below:

$$L_I = \frac{528\,ft}{2} = 264\,ft$$

$$\theta = Sin^{-1}\left(\frac{1.2\,ft}{264\,ft}\right) = 0.26$$

$$L_f = 264\frac{264\,ft}{\cos(0.39)} = 264.003$$

$$\varepsilon = \frac{264.004\,ft - 264\,ft}{264\,ft} = \frac{0.003\,ft}{264\,ft} \times 100 = 0.001\%$$

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GSE HyperFrictionFlex*

Textured HyperFlex* HDPE Geomembrane

GSE HyperFrictionFlex is a premium grade, high density polyethylene (HDPE) geomembrane produced from a specially formulated, virgin polyethylene resin, and textured using GSE's patented FrictionFlex* process. The polyethylene resin is designed specifically for flexible geomembrane applications. HyperFlex has outstanding resistance to UV radiation and stress cracking and is therefore highly suited for exposed applications. The FrictionFlex process is the only manufacturing method that provides a textured material without significant reduction of any of the physical properties of the smooth surfaced membrane. No other textured membrane provides an equivalent combination of enhanced slope stability and resistance to containment failure if settlement of the lined structure occurs.

TESTED PROPERTY	TEST METHOD	MINIMUM VALUES			
Thickness, mils (mm)	ASTM D 751/1593/5199	36 (0.90)	54 (1.35)	72 (1.80)	90 (2.25)
Density, g/cm³	ASTM D 792/1505	0.94	0.94	0.94	0.94
Tensile Properties (each direction)	ASTM D 638, Type IV				
Strength at Break, lb/in-width (N/mm)	Dumbell, 2 ipm	162 (28)	243 (43)	324 (57)	405 (71)
Strength at Yield, lb/in-width (N/mm)		86 (15)	130 (23)	173 (30)	216 (38)
Elongation at Break, %	G.L. = 2.5 in (64 mm)	500	560	560	560
Elongation at Yield, %	G.L. = 1.3 in (33 mm)	13	13	13	13
Tear Resistance, lb (N)	ASTM D 1004	30 (133)	45 (200)	60 (267)	75 (334)
Puncture Resistance, lb (N)	FTMS 101, Method 2065	52 (231)	80 (356)	105 (467)	130 (579)
Carbon Black Content, %	ASTM D 1603	2.0	2.0	2.0	2.0
Environmental Stress Crack Resistance, hr	ASTM D 1693, Cond. B	1500	1500	1500	1500

REFERENCE PROPERTY	TEST METHOD	NOMINAL VALUES			
Thickness, mils (mm)	ASTM D 751/1593/5199	40 (1.0)	60 (1.5)	80 (2.0)	100 (2.5)
Roll Length (approximate), ft (m)		665 (216)	470 (215)	350 (107)	280 (85)
Low Temperature Brittleness, °F (°C)	ASTM D 746, Cond. B	<-107 (<-77)	<-107 (<-77)	<-107 (<-77)	<-107 (<-77)
Oxidative Induction Time, minutes	ASTM D 3895, 200 °C Pure O_2, 1 atm	100	100	100	10
Carbon Black Dispersion	ASTM D 3015	A1,A2,B1	A1,A2,B1	A1,A2,B1	A1,A2,B1
Dimensional Stability (each direction), %	ASTM D 1204, 100 °C, 1 hr	±2	±2	±2	±2
Melt Flow Index, g/10 minutes	ASTM D 1238, Cond.190/2.16	≤1.0	≤1.0	≤1.0	≤1.0

GSE HyperFrictionFlex is available in rolls approximately 22.5 ft (6.9 m) and 24 ft (7.3 m) wide and weighing about 3,500 lb (1,588 kg). Other material thicknesses are available upon request. See the FrictionFlex Application Data Sheet for more information regarding the GSE FrictionFlex texturing process.

** Roll lengths correspond to the 24 ft (7.3 m) wide roll goods.

This information is provided for reference purposes only and is not intended as a warranty or guarantee. GSE assumes no liability in connection with the use of this information. Check with GSE for current, standard minimum quality assurance procedures.

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DS 002 R03/04/98

Appendix B

Geotextile Puncture Resistance

Required Puncture Resistance of Geotextile

OBJECTIVE: Determine the puncture resistance of the cushion geotextile required to prevent puncturing of the geomembrane by gravel layer

METHOD: The cushion geotextile is intended to protect the geomembrane from punctures caused by the gravel layer above the geomembrane. The required puncture resistence for the cushion geotextile will be dictated by the largest particle diameter for the gravel layer under the force provided by the waste and cover system above. The required puncture resistance was solved for using the formula presented in "Designing with Geosynthetics" 3rd edition, Koerner, 1995, pg 165 (see page 5 of this appendix). The formula is given below.

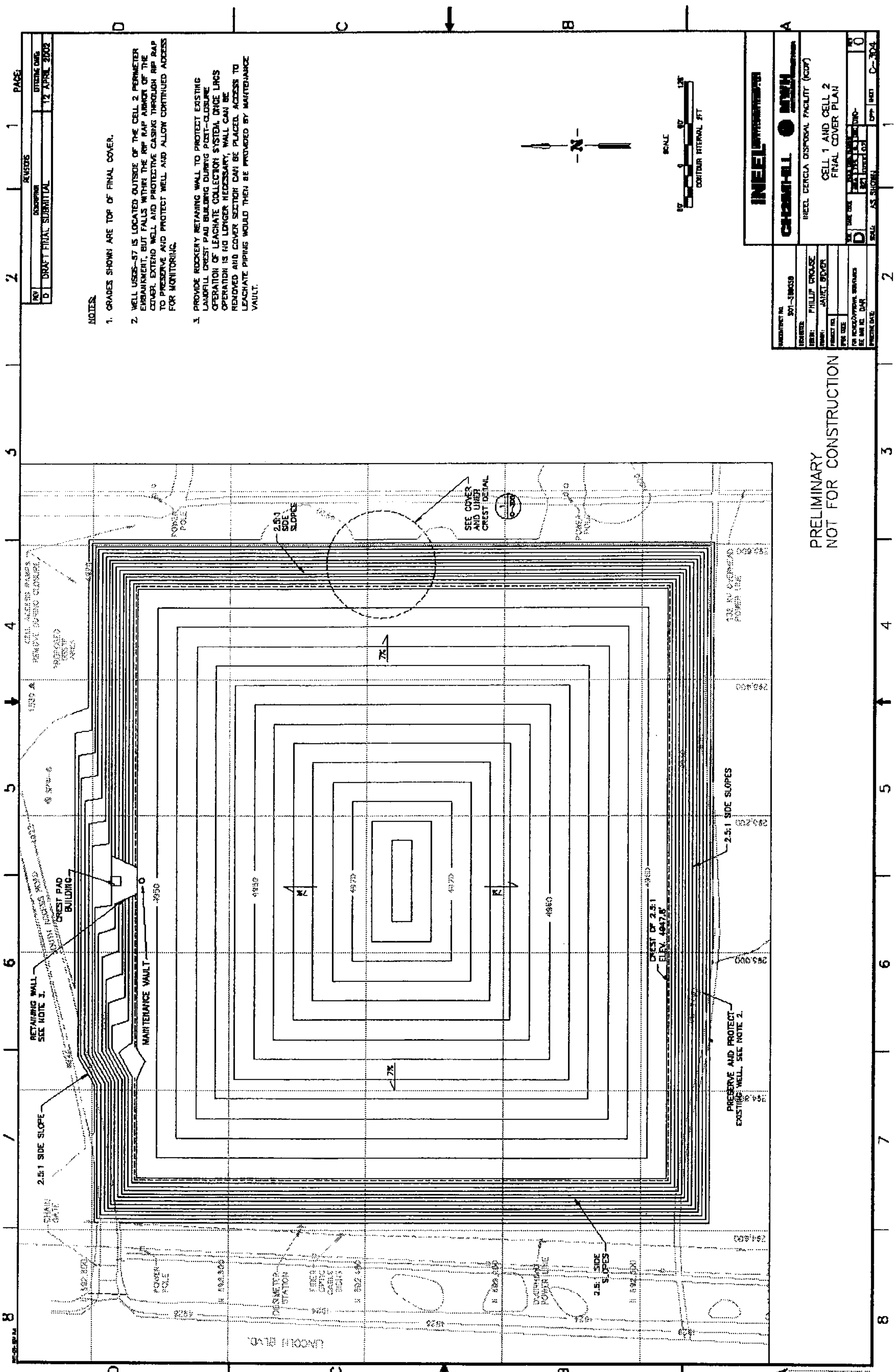
$F_{req} = p' * d_a^2 * S_1 * S_2 * S_3$

F_{req} - required vertical force to be resisted
p' - pressure exerted on geotextile
d_a - average diameter of puncturing aggregate
S_1 - protrusion factor, h_h/d_a
h_h - protrusion height <= d_a
S_2 - scale factor to adjust ASTM D4833 test value using 5/16" diameter rod to the actual puncturing object = d_{probe}/d_a
d_{probe} - probe diameter which is 5/16" for ASTM D4833
S_3 - shape factor to adjust test puncture probe of ASTM D4833 to actual shape of puncturing object = $1 - A_p/A_c$
(values range from 0.8 for round sand, to 0.7 for run-of bank gravel, to 0.4 for crushed rock, to 0.3 for shot rock)
A_p - projected area of particle
A_c - area of smallest circumscribed circle

Calculation:

Maximum elevation of cover system:	4974	ft	See pg 2 drawing C-304
Minimum elevation of liner system:	4884	ft	See pg 3 drawing C-301
Maximum waste thickness:	90	ft	
Estimated waste density:	133.5	pcf	from EDF-ER-266
Average aggregate diameter (d_a):	0.67	in	17mm = 0.67 in for Gravel D_{50}
Probe diameter (d_{probe}):	0.3125	in	
A_p/A_c ratio:	0.4		conservate assumption, see pg. 5
Maximum anticipated pressure on the liner system (p'):	83	psi	
Protrusion factor (S_1):	0.50		assume $h_h = 0.5\ d_a$,
Scale factor (S_2):	0.47		
Shape fator (S_3):	0.6		
F_{req}:	5.2	lbs	

	Installation Damage	Creep	Chemical Degradation	Bio Degradation
Typ Cushion Reduction Factors Ranges (Koerner,p. 149)	1.1 to 2.5	1.5 to 2.0	1.0 to 2.0	1.0 to 1.2
Reduction Factors Used:	2	2	1.5	1.1
Total RF:	6.6			
Geotextile Specified Puncture Resistance:	135 lbs			
F_{allow}:	20.5 lbs			
$FS = F_{allow}/F_{req}$:	3.9			



(See Drawing C-303 in the ICDF Draft Final Drawings for further detail.)

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CHLM #1 Calcs

Safety Factor Against Geomembrane Puncture - Design Calculator

(1) uses $MF_S = 0.5 \rightarrow$ more conservative than EPA

(2) uses $H = 15mm = D_{50}$ < conservative than EPA

Problem Statement

There are many circumstances where geomembranes are placed on or beneath soils containing relatively large-sized stones. For example, poorly prepared soil subgrade with stones protruding from the surface, and cases where crushed-stoned drainage layers are to be placed above the geomembrane.

In all of these situations, a nonwoven needle-punched geotextile can provide significant puncture protection to the geomembrane. The issue of determining the required mass per unit area of the geotextile becomes critical.

(3) uses only RF_{CR} and RF_{CBD} factors

The method presented herein (Koerner, 1998) focuses on the protection of 1.5 mm thick HDPE geomembranes. The method uses the design by function approach.

$$FS = \frac{p_{allow}}{p_{act}}$$

where:

FS	factor of safety against geomembrane puncture
P_{act}	actual pressure due to the landfill contents or surface impoundment
P_{allow}	allowable pressure using different types of geotextiles and site specific conditions.

p_{allow} is determined by the following equation:

$$P_{allow} = \left(50 + 0.00045\frac{M}{H^2}\right)\left[\frac{1}{MF_S * MF_{PD} * MF_A}\right]\left[\frac{1}{RF_{CR} * RF_{CBD}}\right]$$

where:

Symbol	Name	Unit
P_{allow}	allowable pressure	kPa
M	geotextile mass per unit area	g/m2
H	height of the protrusion above the subgrade	m
MF_S	modification factor for protrusion shape	-
MF_{PD}	modification factor for packing density	-
MF_A	modification factor for arching in solids	-
RF_{CR}	reduction factor for long-term creep	-
RF_{CBD}	reduction factor for long-term chemical/biological degradation	-

Appendix C

Geomembrane Wind Lift Analysis

Evaporation Pond Wind Lift Analysis

OBJECTIVE: Determine the necessary anchorage to negate HDPE geomembrane movement due to the wind in the evaporation pond. Determine this for both short and long term conditions.

METHOD: Use design tables from Koerner and Wayne, *Effect of Wind Uplift on Liner Systems*, Geosynthetic Fabrics Report, July/August 1988. A wind speed of 70 mph will be used based on EDF-ER-323, Evaporation Pond Berm Overtopping Analysis, which used this value to calculate wave runup within the evaporation pond.

ASSUMPTIONS: Sand bags are asssumed to be 70 lbs each (minimum)

CALCULATIONS:

Short Term: Sand bags will need to be placed on top of the geomembrane liner after installation because the geomembrane is the upper most layer in the evaporation pond liner system.

The short term wind speed of 35 mph was used which is approximately half of 70mph which is the upper end of wind speed gusts measured.

Based on attached Figures 3 - 10, the average Cp (pressure coefficient) is -0.2

Use Table 2(b), and a Cp value of -0.2 to determine sand bag spacing,

25 mph wind - 1 sand bag per 219 ft^2
50 mph wind - 1 sand bag per 54.8 ft^2

Using linear interpolation of these values to determine the area per sand bag for a 35 mph wind, one sand bag should be placed for every 153 square feet.

Sand bags should be tied off every 5 linear feet along the rope. To determine the bag line spacing, divide 153 square feet by 5 feet.

Sand Bag Line Spacing = 153 sq. ft / 5 ft = 30.6 ft or approximately 30 feet

Long Term:

A long term wind speed of 45 mph was used which is approximately 65% of 70 mph which is the upper end of wind speed gusts measured.

Based on attached Figures 3 - 10, the average Cp (pressure coefficient) is -0.2

Use Table 2(b), and a Cp value of -0.2 to determine sand bag spacing,

25 mph wind - 1 sand bag per 219 ft2
50 mph wind - 1 sand bag per 54.8 ft2

Using linear interpolation of these values to determine the area per sand bag for a 45 mph wind, one sand bag should be placed for every 86.2 square feet.

Sand bags should tied off every 5 linear feet along the rope. To determine the bag line spacing, divide 86.2 square feet by 5 feet.

Sand Bag Line Spacing = 153 sq. ft / 5 ft = 17.2 ft or approximately 15 feet

For long term conditions, in addition to sand bagging it is suggested that the bottom of the pond be covered with a minimum of either soil or fluid in the bottom to counterweight additional long term uplift pressures due to wind.

Conclusion: For short term conditions, sand bags should be placed at a 30 feet spacing. For long term conditions, sand bags should be placed every 15 feet and either fluid or soil should be placed in the bottom.

BY MARK H. WAYNE and
ROBERT M. KOERNER

Effect of Wind Uplift on Liner Systems

The installation of geomembrane liner systems for reservoirs, landfills and other related impoundments is greatly affected by wind. Natural winds near the earth's surface are influenced by friction at the ground surface and turbulence within the flowing air mass. Uplift forces develop as a result of wind flow separation, which occurs when the air mass decelerates or when boundary shapes are irregular. Downstream from the point of air flow separation, a disturbed flow, or wake of turbulent eddies is formed and flow reverses. When such uplift forces are not adequately resisted by (a) dead weight (sandbags, tires, etc.), (b) suction devices (installation of suction cowls as used in the agricultural industry) or (c) permanent bonding (e.g., placement of an asphalt emulsion) methods, various stresses on the geomembrane sheets and seams are exerted. In the limit they will lift the sheet, and/or cause tearing to occur.

Discussion

This brief paper uses information obtained by Dedrick [1,2] to develop a design methodology that can be used in determining the magnitude and distribution of tractive (uplift) forces on geomembrane systems. In particular the paper focuses on two reservoir sections with the following geometric characteristics, (see Figures 1(a-c)).

Geometric Detail	Section 1 [fig. 1(b)]	Section 2 [fig. 1(c)]
Approach Slope	1:3	1:1
Approach-Slope Height	5D/4	5D/4
Leeward Slope	1:1	1:1
Breadth-to-Depth Ratio	5	5

Dedrick [1,2] highlights the required fluid mechanics principles relative to modeling of water-harvesting catchments and reservoirs for agricultural purposes. This same information can be applied to solid waste land disposal cells and surface impoundments during construction and prior to filling that have site geometry similar to the previously mentioned sections.

Figure 1(a) illustrates the geometric configuration of the sections studied and also indicates pressure measurement locations for the various sections examined. In Dedrick's study each model section was subjected to test wind velocity ranges of 92 to 96, 126 to 130, and 157 to 160 mph. Additionally, airflow was varied from 0° (which was parallel

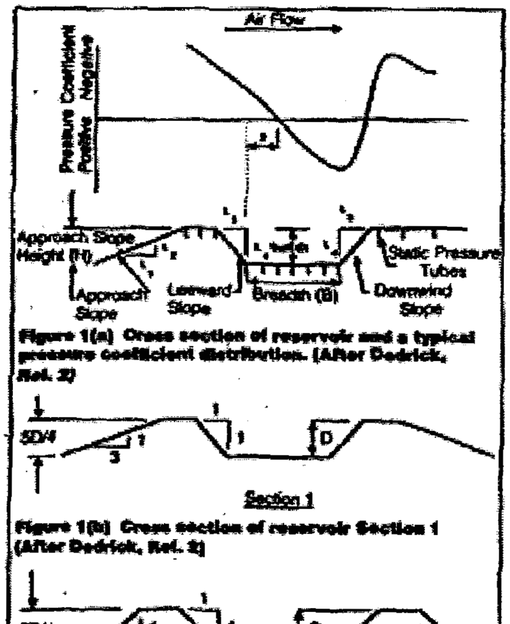

Figure 1(a) Cross section of reservoir and a typical pressure coefficient distribution. (After Dedrick, Ref. 2)

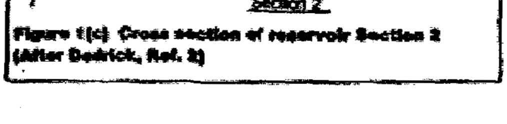

Figure 1(b) Cross section of reservoir Section 1 (After Dedrick, Ref. 2)

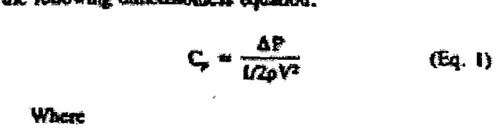

Figure 1(c) Cross section of reservoir Section 2 (After Dedrick, Ref. 2)

Through proper dimensional analysis, Dedrick [1,2] formed the following dimensionless equation.

$$C_p = \frac{\Delta P}{1/2\rho V^2} \qquad \text{(Eq. 1)}$$

Where

C_p = pressure coefficient
ΔP = pressure difference (lbf/ft^2)
ρ = fluid density ($slug/ft^3$ = $lbf\ s^2/ft^4$)
V = air velocity (ft/sec)

Table 2(a) Tractive (uplift) forces, in lb/ft², produced at various wind speeds as associated with C values found in Figures 3–10.

Wind Speed →	25 mph	50 mph	75 mph	100 mph	125 mph
Cp Values					
−0.2	0.32	1.28	2.88	5.11	8
−0.4	0.64	2.56	5.75	10.2	16
−0.6	0.96	3.83	8.63	15.3	24
−0.8	1.28	5.11	11.5	20.5	32
−1	1.6	6.39	14.4	25.6	40

Table 2(b) Sandbag spacing requirements in ft², necessary to compensate for the uplift forces found in Table 2(a).

Wind Speed →	25 mph	50 mph	75 mph	100 mph	125 mph
Cp Values					
−0.2	219	54.8	24.3	13.7	8.8
−0.4	109.5	27.4	12.2	6.8	4.4
−0.6	73	18.3	8.1	4.6	2.9
−0.8	54.8	13.7	6.1	3.4	2.2
−1	43.8	11	4.9	2.7	1.8

value of uplift pressure can then be resisted by whatever mechanism is available, e.g., tires or sandbags.

To illustrate use of the preceeding information, we have developed example design tables and an illustrative problem. Table 2(a) has been developed on the basis that air density is conservatively taken at sea level ($\rho = 0.002377$ slug/ft³) and wind speeds are taken as 25, 50, 75, 100, and 125 mph. Calculated sandbag spacings were based on the use of 70 lb sandbags and are presented in Table 2(b).

Note that these sandbag spacing are not required over the entire site but only where negative values of C_p exist. Thus for the section illustrated in Figure 3, with a design wind velocity of 75 mph, uplift pressures will develop as shown in Figure 11(a). To resist such forces, sandbags weighing 70 lb each would be required as shown in Figure 11(b).

Conclusion

The problem of wind forces lifting up liners and occasionally tearing them and/or their seams, is addressed in this short paper. While its major focus is toward geomembranes, other geosynthetics have experienced wind-related construction problems. For example, geotextiles used to retard reflective cracking in highway rehabilitation and geotextiles used on slopes for erosion control systems have been known to be problematic in high wind locations. The techniques presented should apply to these situations as well as with geomembranes.

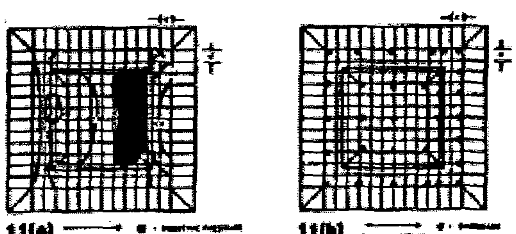

Figure 11(a) Induced uplift pressures, in psf, produced by 75 mph winds. (b) Sandbag spacing required for geomembrane system illustrated in (a). Note that dimensions are exaggerated for illustrative purposes.

References

1. Dedrick, A. R., Air Pressures Over Surfaces Exposed to Wind. I. Water Harvesting Catchments, TRANSACTIONS of the ASAE, Vol. 17, No. 5, 1974.
2. Dedrick, A. R., Air Pressures Over Surfaces Exposed to Wind.-II. Reservoirs. TRANSACTIONS of the ASAE, Vol. 18, No. 3, 1975.
3. Richardson, G. N. and Koerner, R. M., Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments, United States Environmental Protection Agency, Contract No. 68-03-3338.
4. Fox, R. W. and McDonald, A. T., Introduction to Fluid Mechanics, 3rd ed., John Wiley and Sons, New York, 1985.

Acknowledgements

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The Tensar Corporation
Fluid Systems Inc./National Seal Co.

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Appendix D

Anchor Trench Pullout Resistance Calculation

ANCHOR TRENCH PULL OUT RESISTANCE CALCULATION

Objective: Determine Anchor Trench Pull Out Resistance

Input:

Minimum Liner interface friction angle (d):	29.3	°	(Nonwoven GCL/composite drainage net, see attached shear strentgh information from EDF-ER-268)
Minimum Liner interface adhesion (a):	0	psf	(Nonwoven GCL/composite drainage net, see attached shear strentgh information from EDF-ER-268)
Cover Soil Density (g_{cs}):	120	pcf	
Anchor Trench Soil Density (g):	110	pcf	
Anchor Trench Width (L_{AT}):	2	ft	
Anchor Trench Depth (d_{AT}):	2	ft	
Operations Layer Thickness (d_{cs}):	3	ft	
Runout Trench Length (L_{RO}):	3	ft	
Anchor Trench Backfill Friction Angle (f):	30	°	

Anchor Trench Runout Resistance

$Pr_1 = (s_{n1} \tan d + a)\, L_{RO}$

$s_{n1} = g_{cs}\, d_{cs}$

$s_{n1} =$ 360 psf

$Pr_1 =$ 606.068 lbs/ft

Anchor Trench Bottom Resistance

$Pr_2 = (s_{n2} \tan d + a)\, L_{AT}$

$s_{n2} = (g_{cs}\, d_{cs}) + (g\, d_{AT})$

$s_{n2} =$ 580 psf

$Pr_2 =$ 650.96 lbs/ft

Anchor Trench Sidewall Resistance

$k_o = (1 - \sin f)$

$k_o =$ 0.5

Average normal stress through anchor trench

$s_{avgv} = (s_{n1} + s_{n2})/2$

$s_{avgv} =$ 470 psf

$s_{havg} = k_o * s_{avgv}$

$s_{havg} =$ 235 psf

$s_{havg} = s_{n3}$

$Pr_3 = (s_{n3} \tan d + a)\, d_{AT}$

$Pr_3 =$ 263.75 lbs/ft

Total Anchor Trench Pullout Resistance

$Tr = P_{r1} + P_{r2} + P_{r3}$

$Tr =$ 1520.78 lbs/foot

Conclusion:
Anchor trench pullout capacity of 1521 lb/ft is greater than the 440 lb/ft that is required to maintain a slope stability safety factor for Case 1 presented in EDF-ER-268. This case is disussed in EDF-ER-268.

ICDF - Interface Strength Values for Veneer Stability Analysis
(EDF-ER-268, Section 3)

Lining System Interface	Strength calc. from regression analysis[a]	Site-Specific Test Results[b]		
	Friction Angle (deg)	Friction Angle (deg)	Apparent Cohesion (psf)	Effective Friction Angle[c] (deg)
Ops Soil/Composite Drainage Net (CDN)	38.0	38.6	86	44.5
CDN/Textured HDPE	24.0	17.8	220	37.5
Textured HDPE/GCL	26.0	27.6	279	47.0
GCL/CDN	19.0[d]	29.3	0	29.3
Textured HDPE/CCL (Soil-Bentonite)[f]	25.0[e]	30.8	129	40.7

Notes:

[a] - See regression graphs in Appendix A, EDF-ER-268; cohesion = 0 psf in regression analysis

[b] - Testing by Precision Geosynthetics (6/01) on site-specific lining materials; normal stress - 100, 250 and 500 psf

[c] - Calculated at normal stress of 500 psf with Cohesion = 0 psf

[d] - 19 deg calc from data, however 24 deg (based on MW tests) used in analysis; see p. 3.3 in EDF-ER-268

[e] - from CHRLF data (1998) see p. 3.3 in EDF-ER-268

[f] - Site-specifc test data on Soil-bentonite compacted to 87% modified; tests currently being rerun at 92%

Appendix E

Water Erosion of Final Cover Surface

WATER EROSION OF SOIL COVER

OBJECTIVE: Determine the cover soil erosion due to surface sheet erosion.

METHOD: The Modified Universal Soil Loss Equation (MUSLE) was used to calculate the average annual soil erosion resulting from sheet flow across the top surface of the final cover. This is the method presented in NUREG/CR-4620 for use at uranium mining tailing impoundments with a 1000 year design life. This method estimates runoff based on rainfall intensity, soil type, length and slope of the surface, and a control factor which represents vegetative and mechanical factors. The equation is given below.

A = R*K*LS*VM

where:
A = the computed loss per unit area in tons per acre per year with the units selected for K and R properly selected
R = the rainfall factor which is the number for rainfall erosion index units plus a factor for snowmelt, if applicable
K = the soil erodibility factor, which is the soil loss rate per erosion index unit for a specified soil as measured on a unit plot that is designed as a 72.6 foot length of uniform 9% slope continuously maintained as clean tilled fallow
LS = the topographic factor, which is the ratio of soil loss from the field slope length to that from a 72.6 foot length under other wise identical conditions.
VM = the dimensionless erosion control factor relating to vegetative and mechanical factors. This factor replaces the cover management factor (C) and the support factor (P) of the original USLE.

CALCULATIONS:

Ryegrass Flats and WRRTF Borrow Soil Areas
(see attached particle size distribution)

Sample #	Percent silt and very fine sand
#1-O	94
#1-O,#2	93
#1-P,#1	80
#1-P,#2	95
#1-Q,#1	94
#1-Q,#2	96
#3-O,#1	58
#3-O,#2	90
#3-P,Alt. #1	90
#3-P, Alt. 2	92
#3-Q,#1	92
#3-Q,#2	80
Maximum	96
Average	87.83

Ryegrass Flats and WRRTF Borrow Soil Areas
(see attached particle size distribution)

Sample #	Percent Sand (0.1 - 2.0 mm)
#1-O	6
#1-O,#2	7
#1-P,#1	20
#1-P,#2	5
#1-Q,#1	6
#1-Q,#2	4
#3-O,#1	42
#3-O,#2	10
#3-P,Alt. #1	10
#3-P, Alt. 2	8
#3-Q,#1	8
#3-Q,#2	20
Maximum	42
Average	12.17

K = 0.7 (use attached nomograph to determine K)
R = 20 (use attached figure 5.3 to determine R)

$$LS = \frac{650 + 450s + 65s^2}{10{,}000 + s^2} \cdot \frac{L^m}{72.6}$$

where:
s = slope steepness in percent
m = exponent dependent upon slope steepness
L = slope length in feet

s = 7 (see attached drawing)
m = 0.5 (see attached Table 5.2)
L = 434 ft (see attached drawing)
LS = 2.077623
VM = 0.18 (average of seeding values shown in table 5.3)

A = 5.235609 tons per acre per year

Determine the thickness of cover erosion per year.

Assume the density of the cover soil is 110 pcf

Erosion =	0.002185	ft per year
Design Life Erosion =	2.185328	ft per 1000 years
	26.22	in per 1000 years
	66.61	cm per 1000 years

CONCLUSION: Overbuild the cover thickness by at least 73 cm to compensate for the erosion estimated over the 1000 year landfill service life.

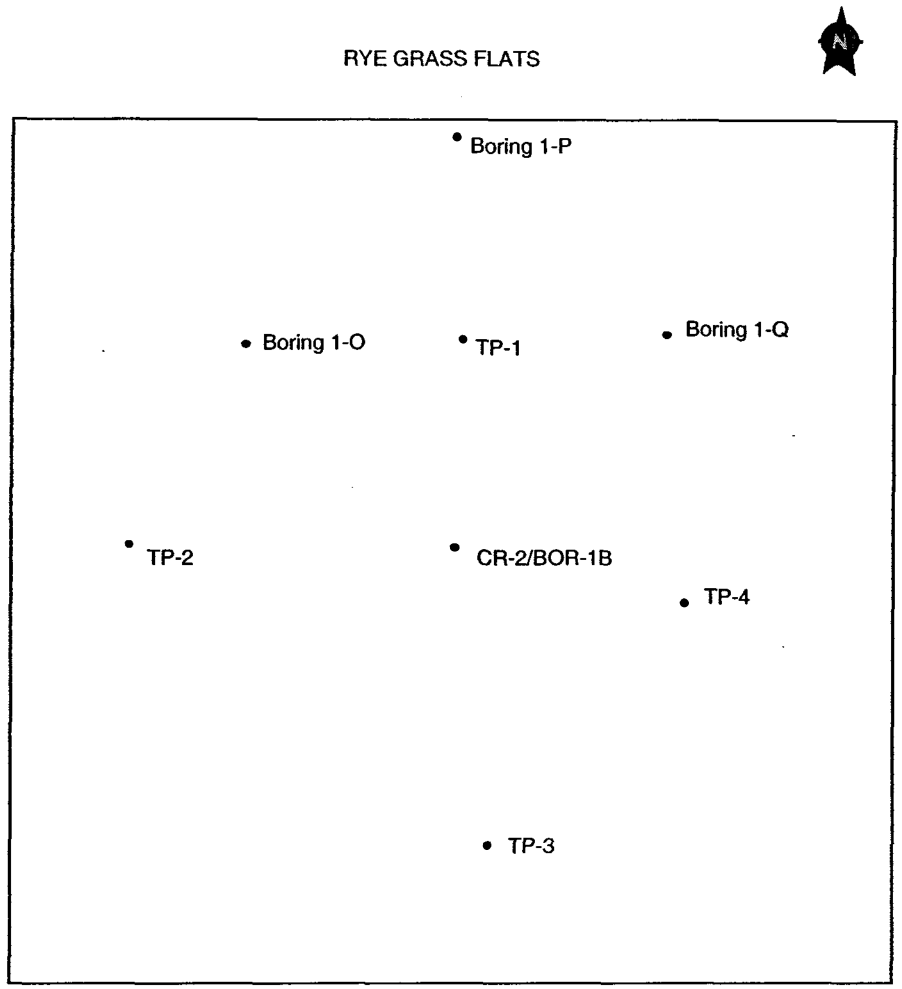


Figure 4-4. Rye Grass Flats boring and test pit locations.

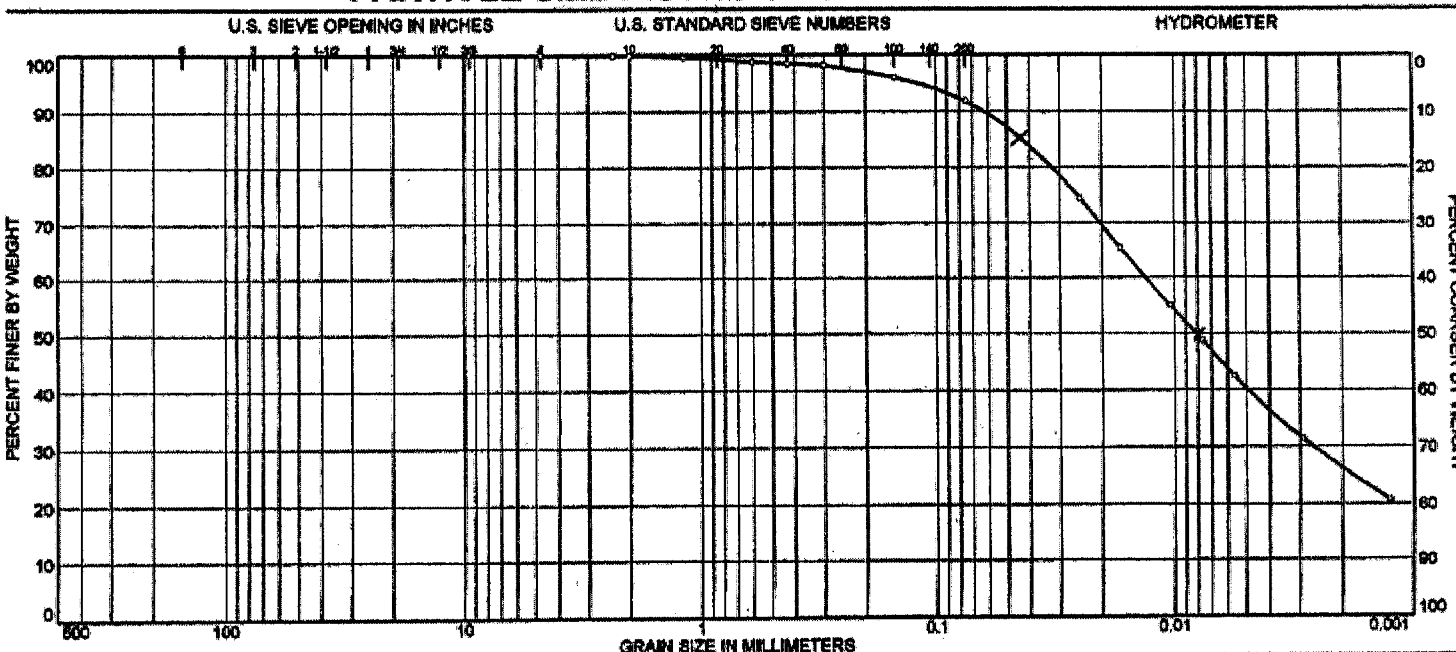
Table 4-1. *(continued).*

Boring/ Sample	Depth	%> 3"	% Gravel Course	% Gravel Fine	% Sand Coarse	% Sand Med	% Sand Fine	% Fines Silt	% Fines Clay	Material Description	Max Dry Density	Opt. Moist %	USCS	NM %	LL	PL	Hydraulic Conductivity
Rye Grass Flats Site																	
1-O #1	2'-2' – 6"	0.0	0.0	0.1	0.1	1.3	6.6	51.6	40.3	Lean clay	106 pcf	18%	CL	10.4	33	18	@1'-6"-4' 5.23E-08 cm/sec Compaction 98.2% Moisture content 16.3%
1-O #2	6' – 7'	0.0	0.0	0.0	0.0	0.1	7.3	43.6	49.0	Lean clay	105 pcf	20%	CL	11.3	33	16	@6'-8' 3.02E-07 cm/sec Compaction 98.8% Moisture content 20.8%
1-P #1	2'-6" – 3'-6"	0.0	0.0	0.0	0.2	6.1	15.8	44.6	33.3	Lean clay with sand	111 pcf	16%	CL	9.2	28	18	Not tested
1-P #2	7' – 8'	0.0	0.0	0.3	0.2	1.3	4.2	47.3	46.7	Lean clay	107 pcf	19%	CL	11.4	38	17	Not tested
1-Q#1	2' – 3'	0.0	0.0	0.0	0.1	1.2	5.8	37.5	55.4	Lean clay	104 pcf	19%	CL	12.7	44	16	@ 1'-3' 1.40 E-07 cm/sec Compaction 98.6% Moisture content 18.1%
1-Q #2	5' – 6'	0.0	0.0	0.3	0.4	0.6	3.4	39.4	55.9	Lean clay	110 pcf	20%	CL	12.6	36	19	@ 4'-6"-6'-6" 2.55 E-06 cm/sec Compaction 97.5% Moisture content 14.7%

Table 4-1. *Borrow source geotechnical test results.*

Boring/ Sample	Depth	%> 3"	% Gravel Coarse	% Gravel Fine	% Sand Coarse	% Sand Med	% Sand Fine	% Fines Silt	% Fines Clay	Material Description	Max Dry Density	Opt. Moist %	USCS	NM %	LL	PL	Hydraulic Conductivity
										WRRTF Site							
3-O #1	6" – 1'	0.0	0.0	0.0	0.5	1.1	48.4	13.3	36.7	Sandy silty clay	110 pcf	18%	CL-ML	8.7	20	13	@6"-2'-6" 8.97E-06 cm/sec Compaction 93.7% Moisture content 17.6%
3-O #2	5'-6" – 6'	0.0	0.0	0.0	0.0	0.3	13.4	12.2	74.1	Lean clay	106 pcf	20%	CL	9.7	39	14	@5'-7' 4.36E-08 cm/sec Compaction 100.4% Moisture content 19.9%
3-Pult #1	6" – 1'	0.0	0.0	0.0	0.0	0.5	13.5	20.9	65.1	Lean clay	107 pcf	19%	CL	12.2	36	18	@6"-3'-6" 2.03E-07 cm/sec Compaction 92.0% Moisture content 18.8%
3-Pult #2	5'-5' – 6"	0.0	0.0	0.0	0.0	0.1	10.4	20.9	68.6	Lean clay	112 pcf	18%	CL	9.2	34	18	@5'-7' 4.10E-07 cm/sec Compaction 95.3% Moisture content 17.0%
3-Q #1	2'-6" – 3'	0.0	0.0	0.0	0.0	0.2	9.3	19.8	70.7	Lean clay	114 pcf	16%	CL	11.1	37	17	Not tested
3-Q #2	7'-6" – 8'	0.0	0.0	0.0	0.0	0.3	28.4	18.9	52.4	Lean clay with sand			CL	8.0	28	14	Not tested

PARTICLE SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.1	0.1	1.3	6.6	51.6	40.3

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
Ryegrass Flats Area	#1-O	2.0' - 2.5'	5/31/00	CL	Lean clay	10.4%	33	18

Client Tom Borschel BBWI

Project INEEL CERCLA Disposal Facility (ICDF)

Project No. 3XD710130 **Plate**

INEL MATERIALS LAB

o Sampled from Core #1, core length - 0'-2.5'. Samples collected by others.

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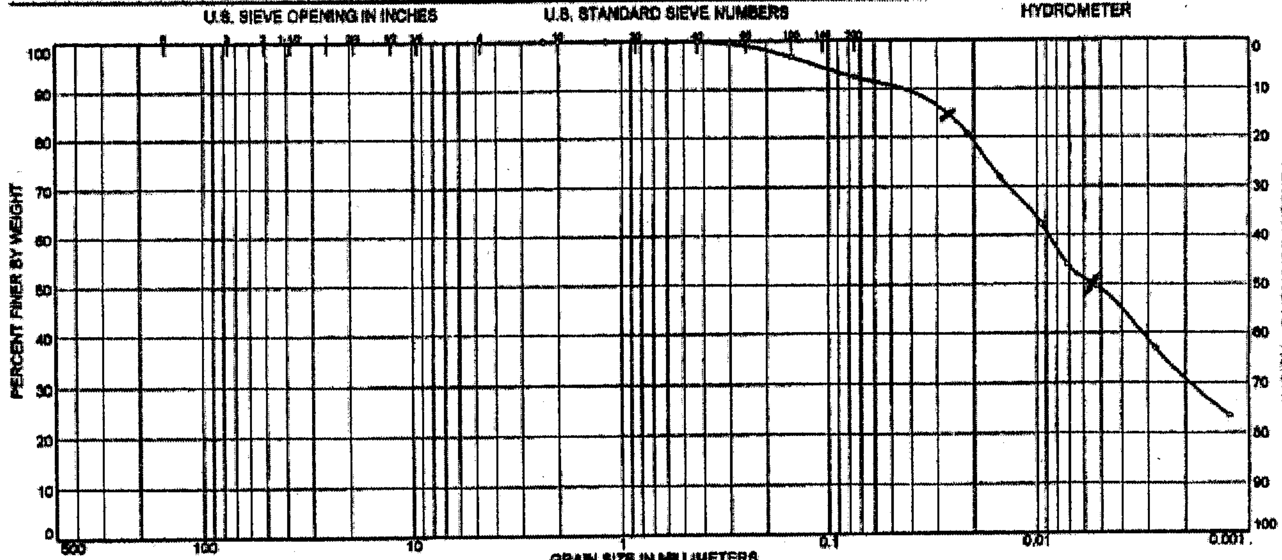

% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.1	7.3	43.6	49.0

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
Ryegrass Flats Area	#1-O, #2	6.0' - 7.0'	6/1/00	CL	Lean clay	11.3%	33	16

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○ Sampled from Core #3, Ryegrass area, Core sample length - 5.0' - 7.5'.

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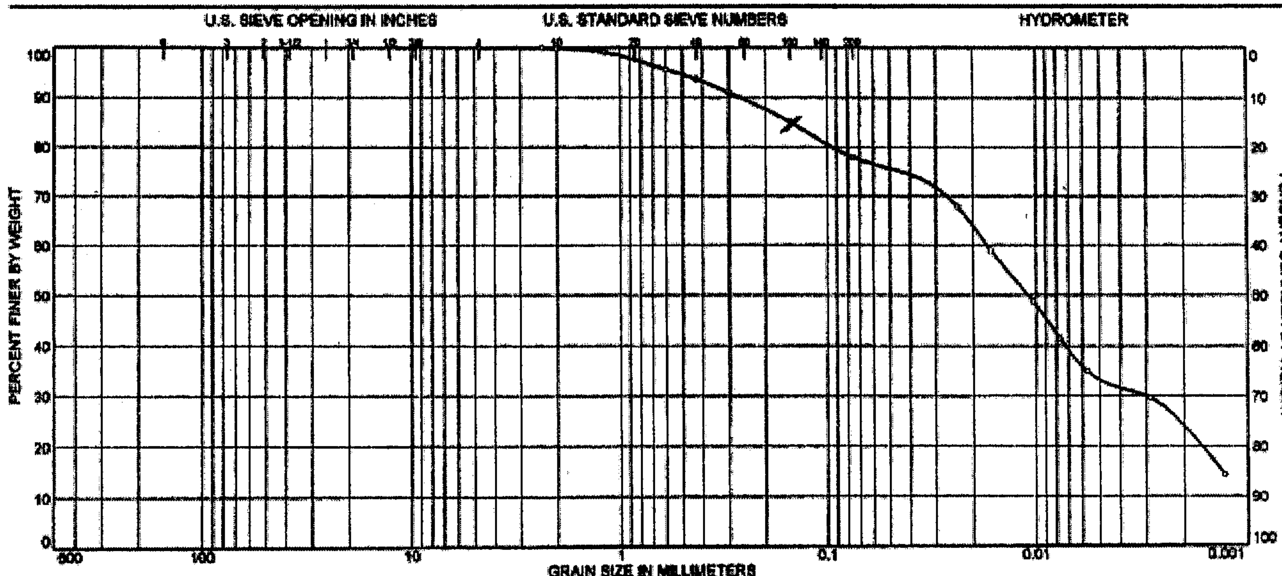

% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.2	6.1	15.8	44.6	33.3

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
Ryegrass Flats Area	#1-P #1	2.5' - 3.5'	6/1/2000	CL	Lean clay with sand	9.2%	28	18

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o Sampled from Core #2, core length - 2.5'-5.0'. Samples collected by others.

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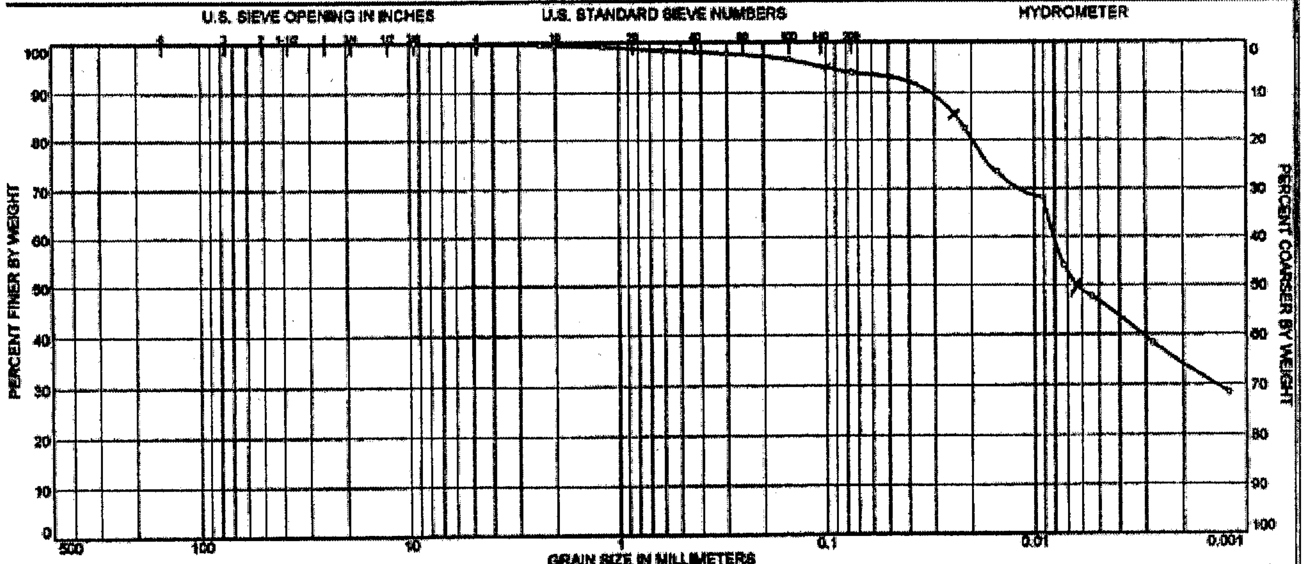

% + 3"	% GRAVEL COARSE	% GRAVEL FINE	% SAND COARSE	% SAND MEDIUM	% SAND FINE	% FINES SILT	% FINES CLAY
0.0	0.0	0.3	0.2	1.3	4.2	47.3	46.7

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
Ryegrass Plain Area	#1-P #2	7.0' - 8.0'	6/15/00	CL	Lean clay	11.4%	38	17

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Plate

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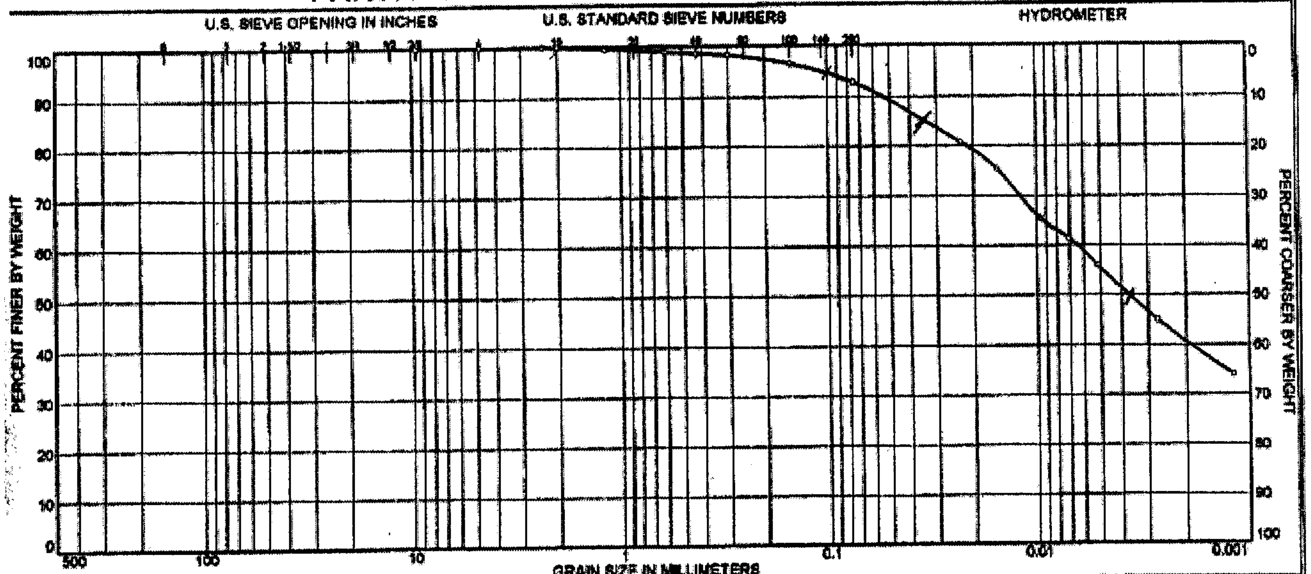
o Sampled from cores #3&4.

PARTICLE SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.1	1.2	5.8	37.5	55.4

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
Ryegrass Flats Area	#1-O #1	2.0' - 3.0'	6/15/00	CL	Lean clay	12.7%	44	16

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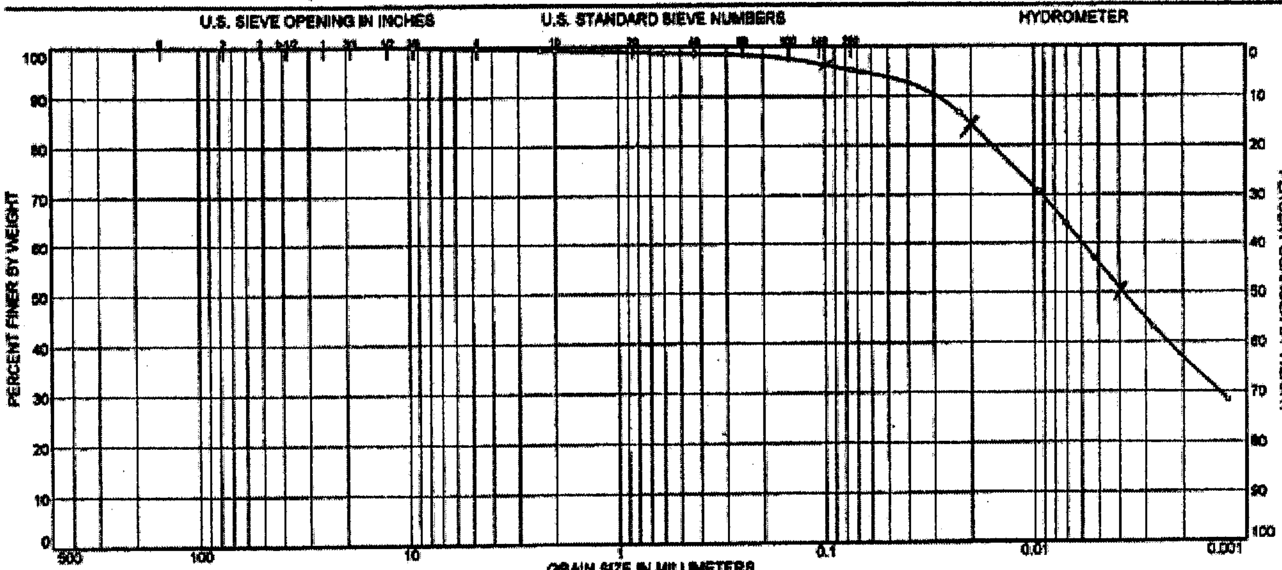
o Sampled from core #1. Samples taken by others.

PARTICLE SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.3	0.4	0.6	3.4	39.4	55.9

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
Ryegrass Flats Area	#1-O #2	5.0' - 6.0'	6/15/00	CL	Lean clay	12.6%	36	17

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INEL MATERIALS LAB

o Sampled from Core #3. Samples taken by others.

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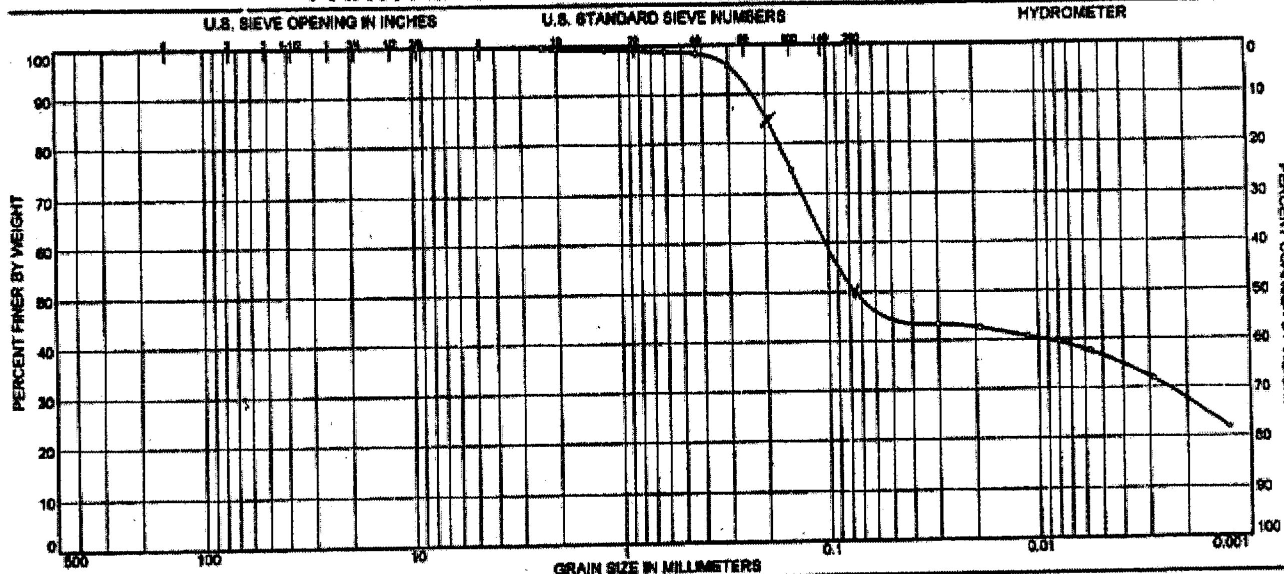

% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.5	1.1	48.4	13.3	36.7

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
WRRTF Area	#3-0 #1	0.5' - 1.0'	6/14/00	CL-ML	Sandy silty clay	8.7%	20	13

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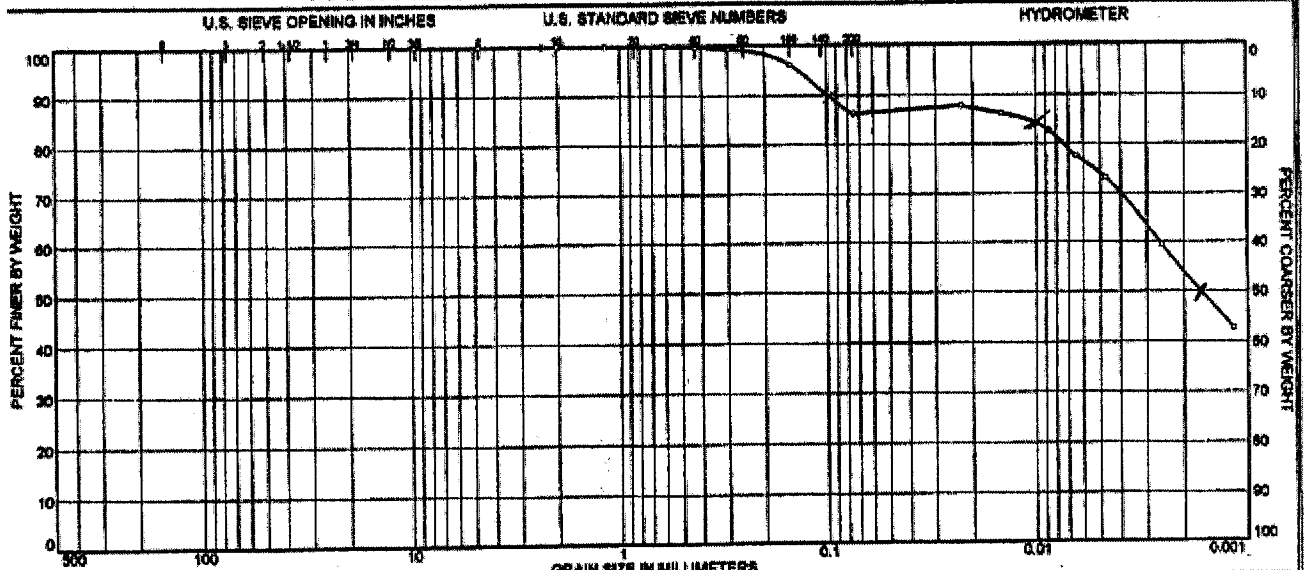
Project No. 3XD710130 Plate

INEL MATERIALS LAB

○ Sampled from WRRTF core #1, Borehole 3-O, core length, 0.0'- 2.5'. Sampled by others.

PARTICLE SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.3	13.4	12.2	74.1

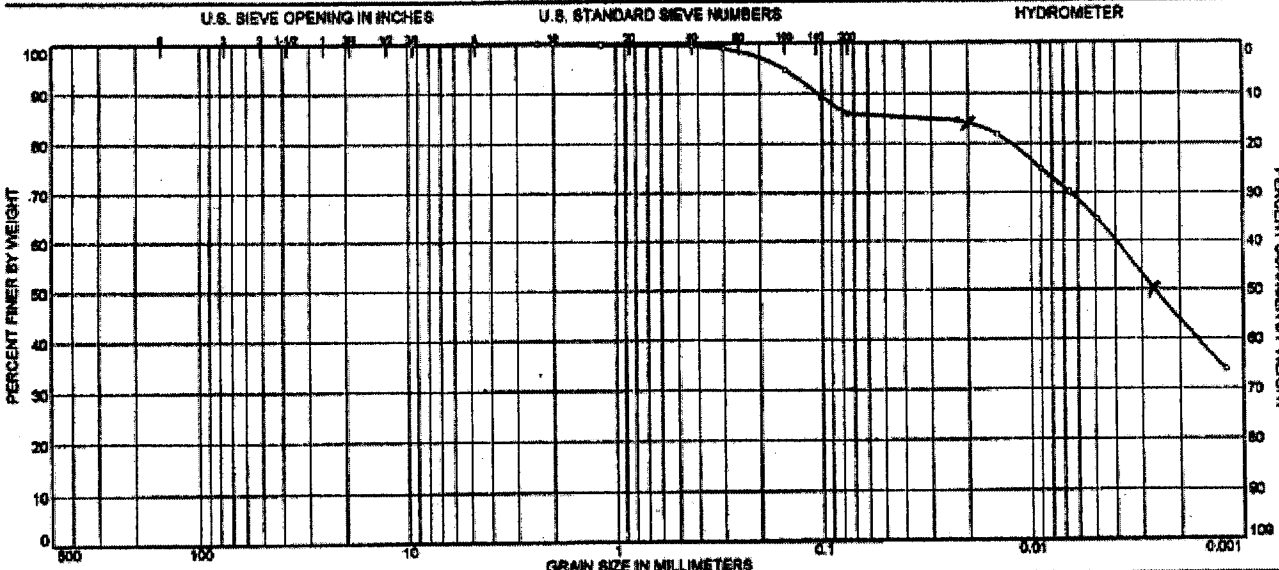
SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
WRRTF Area	#3-O #2	5.5-6.0'	6-14-00	CL	Lean clay	9.7%	39	14

Client Tom Borschel BBWI
Project INEEL CERCLA Disposal Facility (ICDF)
Project No. 3XD710130 Plate

INEL MATERIALS LAB

o Sampled from WRRTF core #3, Borehole #3-O, core length 5.0' - 7.5'. Sampled by others.

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	0.5	13.5	20.9	65.1

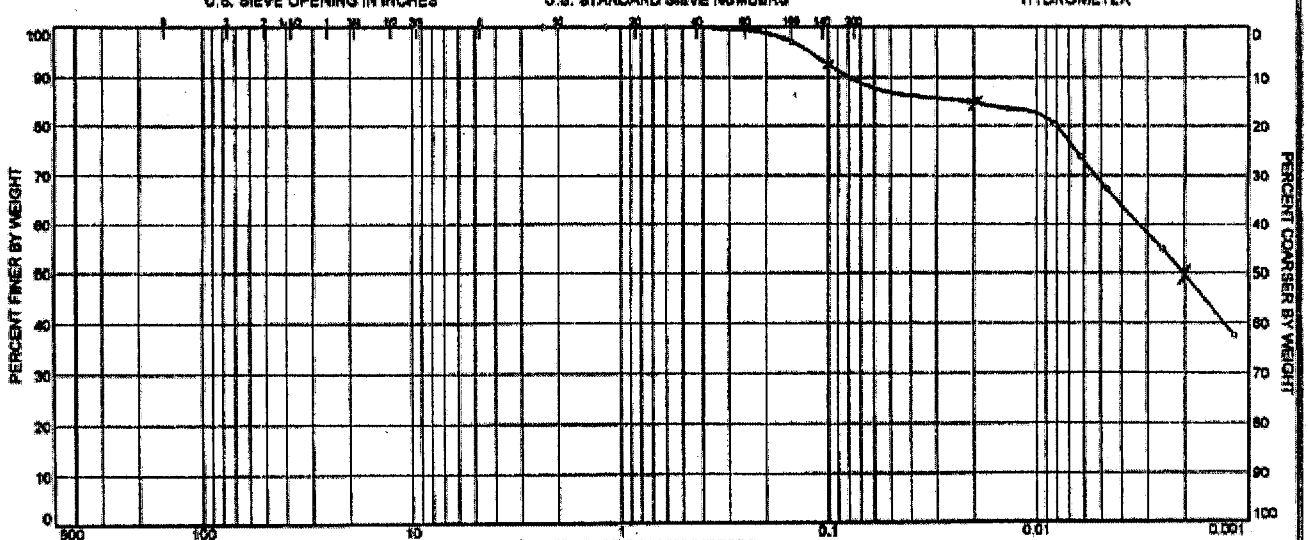
	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
○	WRRTF Area	#3-P Alt. #1	0.5' - 1.0'	6-14-00	CL	Lean clay	12.2%	36	18

Client Tom Borschel BBWI
Project INEEL CERCLA Disposal Facility (ICDF)
Project No. 3XD710130 Plate

INEL MATERIALS LAB

○ Sampled from WRRTF core #1, Borehole #3-P Alt., core length 0.0'-2.5'. Sampled by others.

PARTICLE SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.1	10.4	20.9	68.6

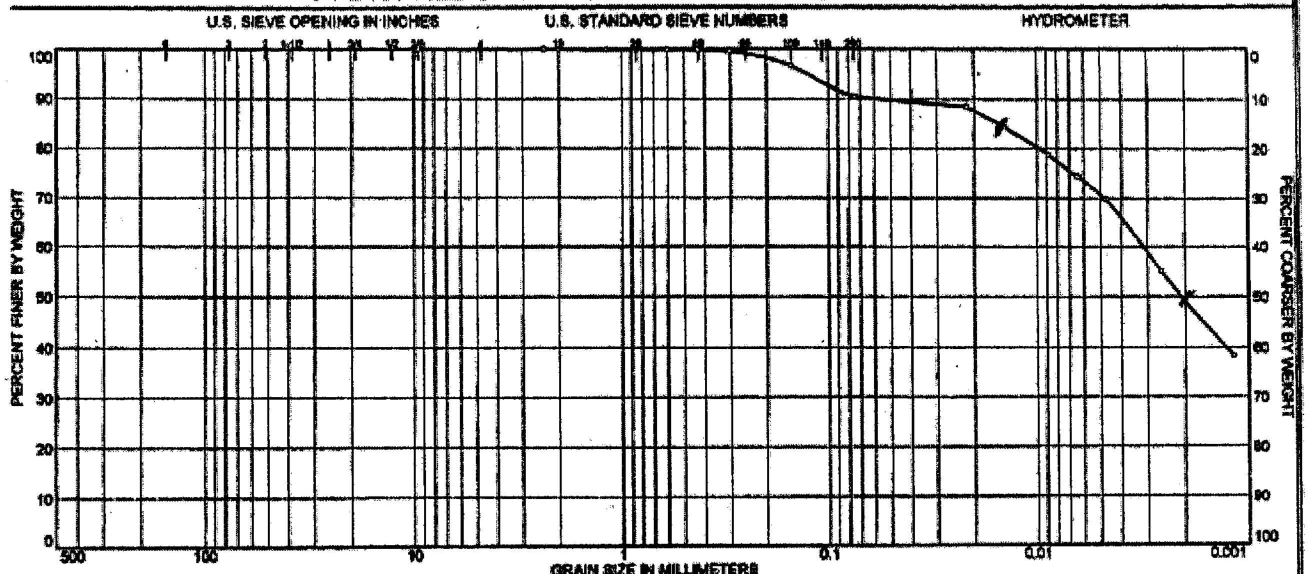
SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
WRRTF Area	#3-P Alt. #2	5.0' - 5.5'	6-14-00	CL	Lean clay	9.2%	34	18

Client Tom Borschel BBWI
Project INEEL CERCLA Disposal Facility (ICDF)
Project No. 3XD710130 Plate

INEL MATERIALS LAB

○ Sampled from WRRTF, core #3, Borehole #3-P Alt.
Sampled by others.

PARTICLE SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL COARSE	% GRAVEL FINE	% SAND COARSE	% SAND MEDIUM	% SAND FINE	% FINES SILT	% FINES CLAY
0.0	0.0	0.0	0.0	0.2	9.3	19.8	70.7

SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
WRRTF Area	#3-Q #1	2.5' - 3.0'	6-19-00	CL	Lean clay	11.1%	37	17

Client Tom Borschel BBWI

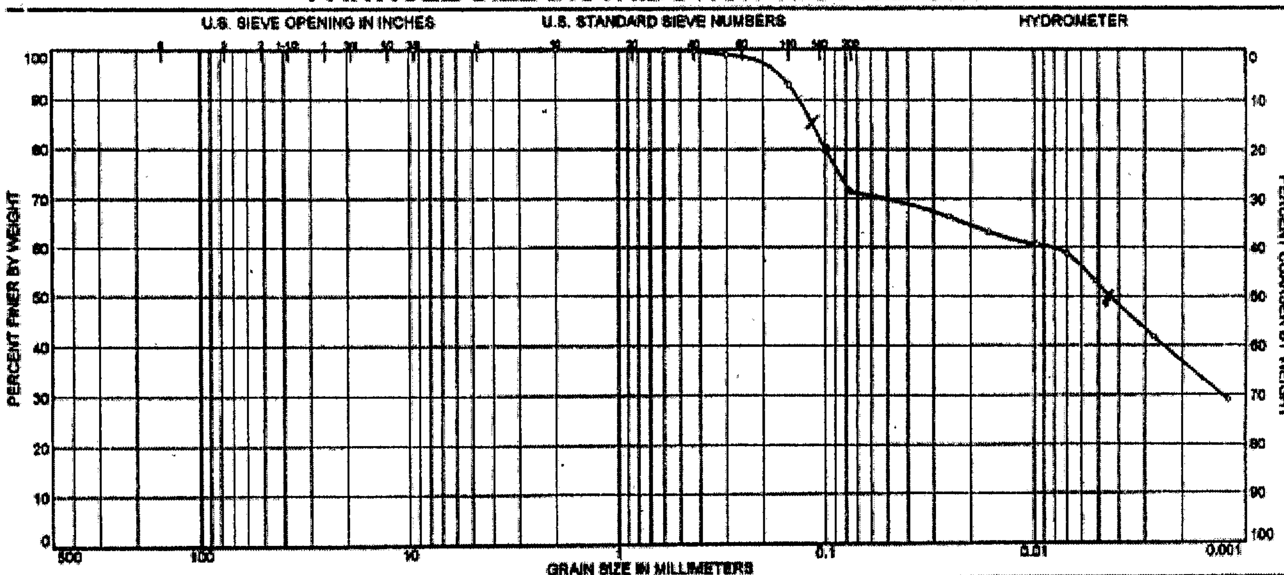
Project INEEL CERCLA Disposal Facility (ICDF)

Project No. 3XD710130 Plate

INEL MATERIALS LAB

○ Sampled from WRRTF, core #2, Borehole #3-Q, core length 2.5'- 5.0'. Sampled by others.

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	0.3	28.4	18.9	52.4

	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
○	WRRTF Area	#3-Q #2	7.5' - 8.0'	6-19-00	CL	Lean clay with sand	8.0%	28	14

Client Tom Borschel BBWI
Project INEEL CERCLA Disposal Facility (ICDF)
Project No. 3XD710130 Plate

INEL MATERIALS LAB

○ Sampled from WRRTF, core #4, borehole #3-Q, core length, 7.5' 10.0'. Sampled by others.

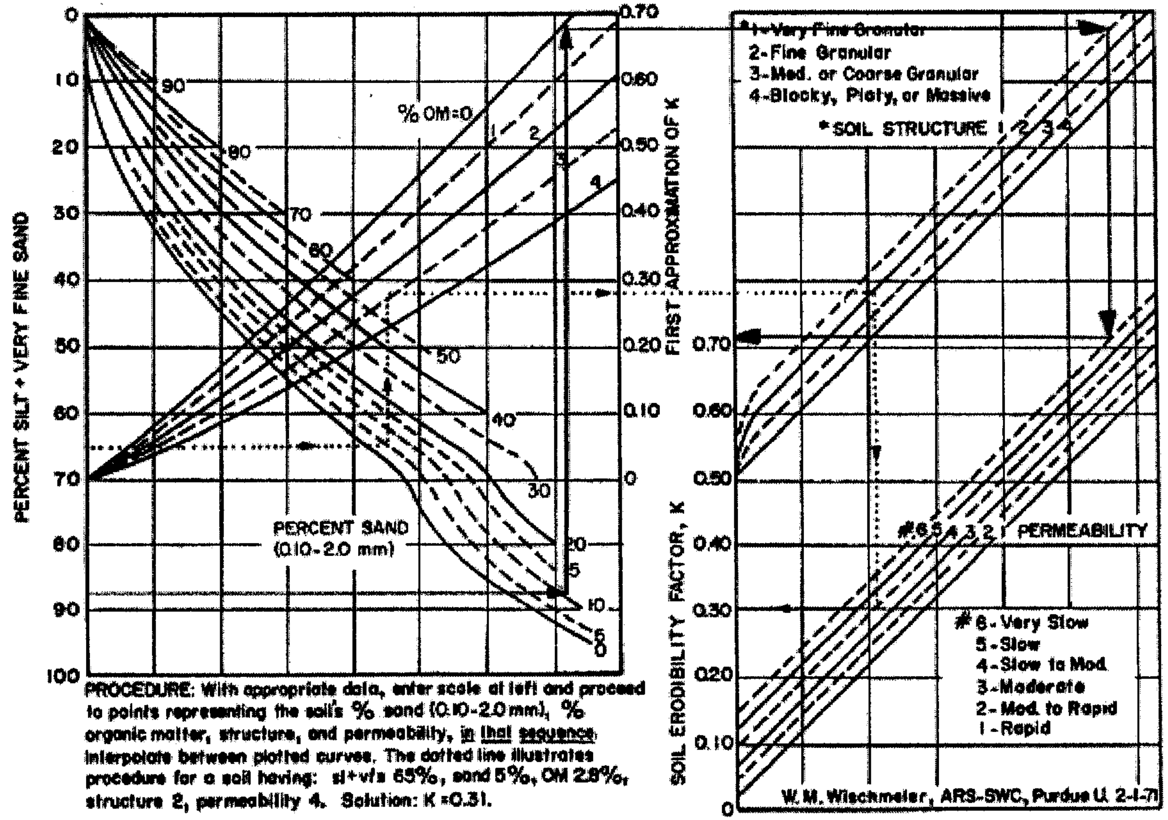
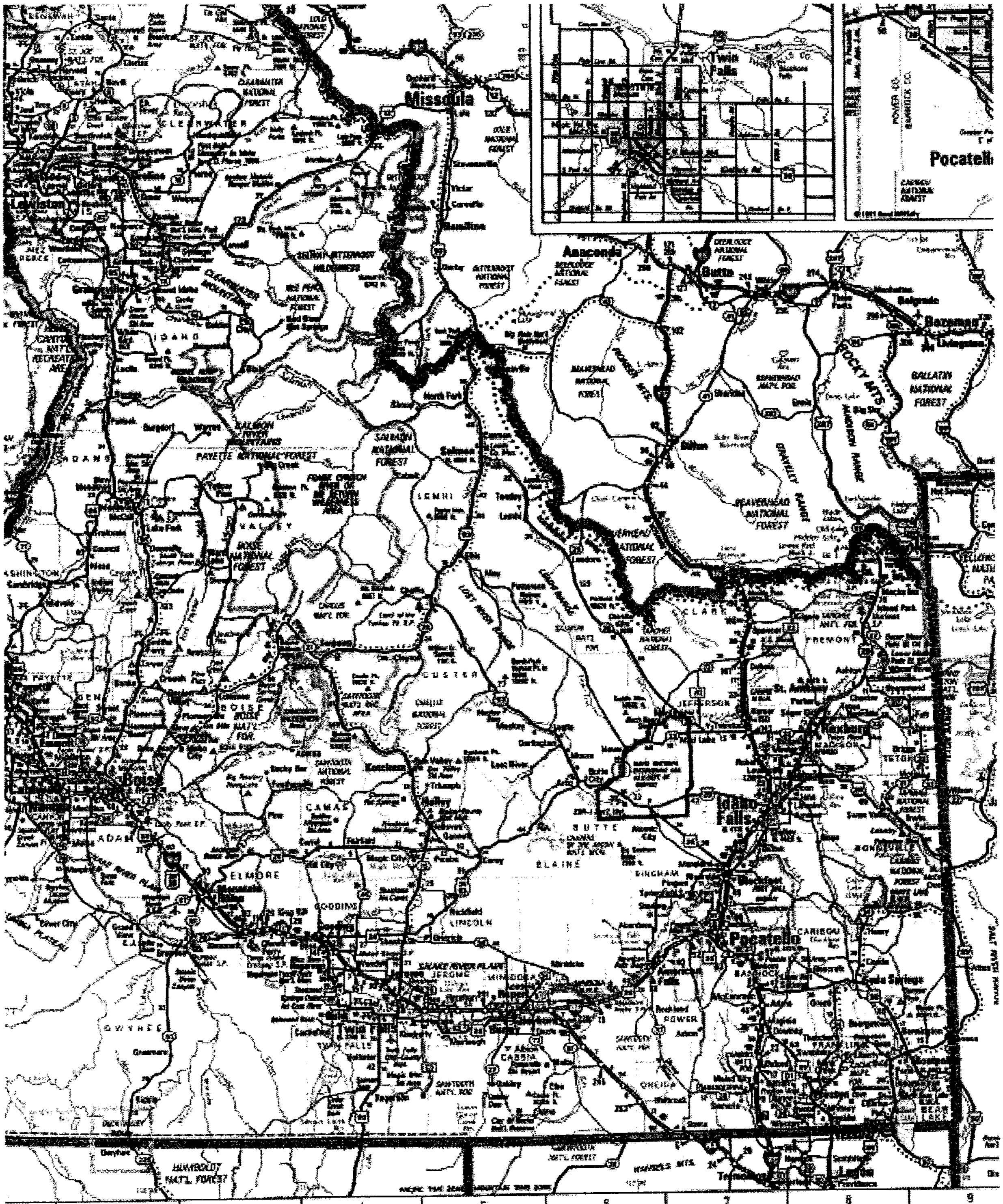


Fig. 5.1. Nomograph for determining soil erodibility factor *K*. *Source:* after Wischmeier et al., 1971.


Missoula
Pocatello
Twin Falls
Anaconda
Butte
Belgrade
Bozeman
Livingston
Lewiston
Boise
Salmon
Pocatello
Idaho Falls
Twin Falls
SALMON NATIONAL FOREST
PAYETTE NATIONAL FOREST
BOISE NATIONAL FOREST
GALLATIN NATIONAL FOREST
CUSTER
LEMHI
CLARK
FREMONT
JEFFERSON
ELMORE
GOODING
JEROME
MINIDOKA
LINCOLN
BLAINE
BUTTE
CAMAS
CARIBOU
POWER
ONEIDA
CASSIA
HUMBOLDT NAT'L FOREST
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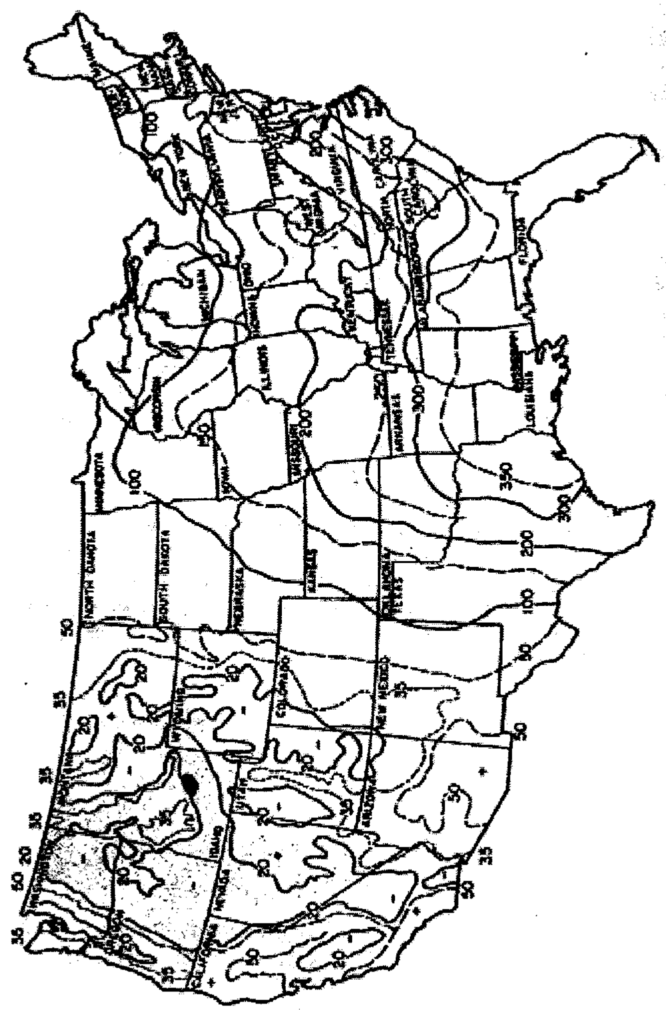
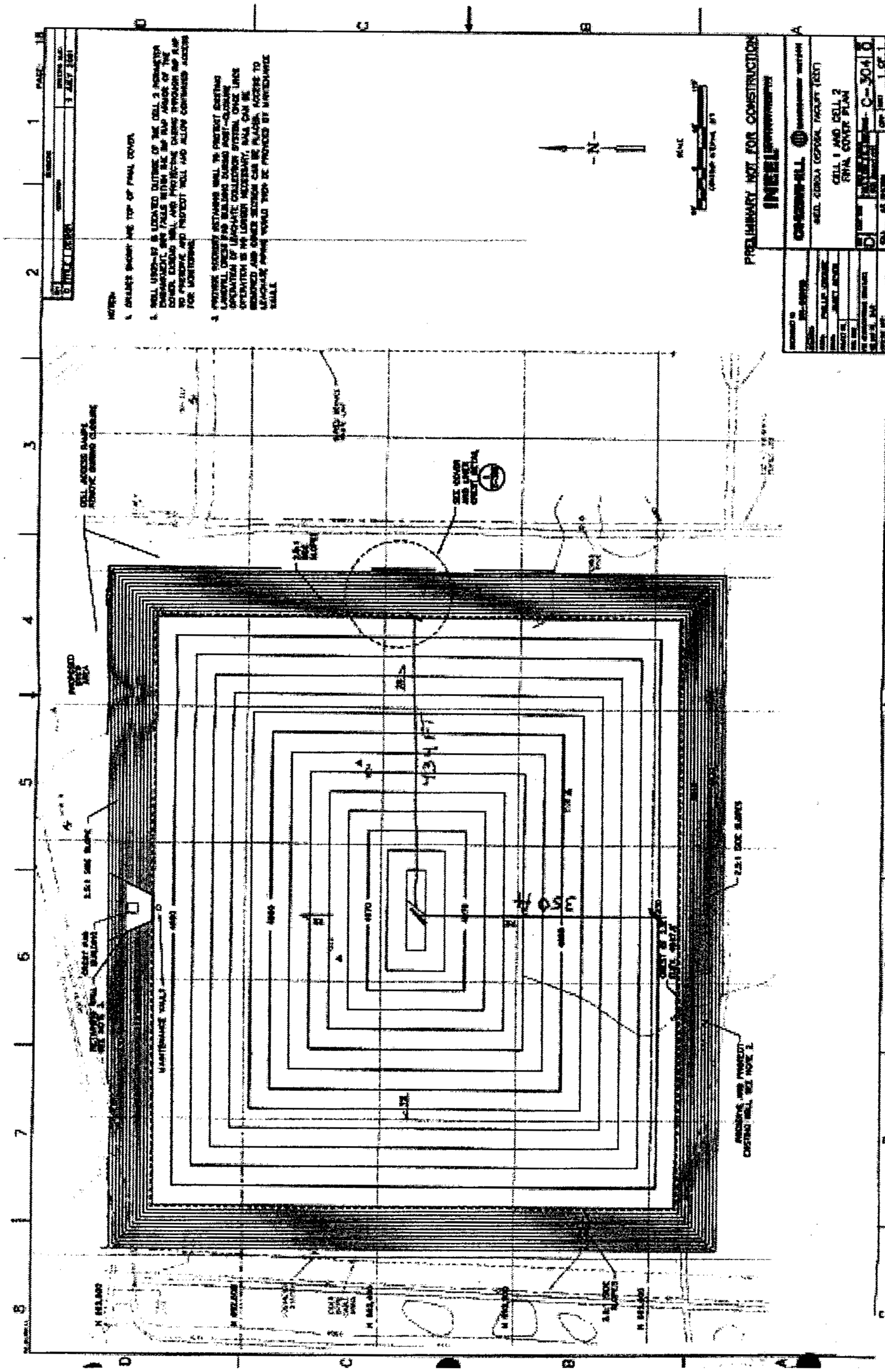


Figure 5.3. Average annual values of the rainfall-erosivity factor, R. (Stewart et al. 1975)



(See Drawing C-304 in the ICDF Draft Final Drawings for more detail.)

5.1.2.3 The Topographic Factor (LS)

Although the effects of both length and steepness of slope have been investigated separately in different research efforts, it is more convenient for analytical purposes to combine the two into one topographic factor, LS. Wischmeier and Smith (1978) developed plots correlating the topographic factor for slopes up to 500 meters in length at slope inclinations from 0.5% up to 50%. Note that flat, short slopes will have less erosion than long, steep slopes and it is to the benefit of the design engineer to optimize slope length and gradients to fit the topography.

The equation to determine the LS factor is as follows:

$$LS = \frac{650 + 450s + 65s^2}{10{,}000 + s^2} \left(\frac{L}{72.6}\right)^m \qquad (5.2)$$

where LS = topographic factor
L = slope length in feet
s = slope steepness in percent
m = exponent dependent upon slope steepness

The slope dependent exponent m is presented in Table 5.2.

Table 5.2 Slope Dependent Exponent

Slope (percent)	m
$s \leq 1.0$	0.2
$1.0 < s \leq 3.0$	0.3
$3.0 < s \leq 5.0$	0.4
$5.0 < s \leq 10.0$	0.5
$s > 10.0$	0.6

5.1.2.4 The VM Factor

The VM factor is the erosion control factor applied in place of the cover and erosion control factors found in the USLE. The erosion control factor accounts for measures implemented at the construction site to include vegetation, mulching, chemical treatments and sprayed emulsions to impede or reduce erosion due to the overland flow of water. Values of the VM factor relative to site-specific conditions are presented in Table 5.3.

The VM factor is perhaps the most sensitive factor to effect the computed erosion loss for a given site. As shown by the values presented on Table 5.3, the development of a permanent vegetative cover can have a significant impact in reducing the computed erosion loss. However, the effectiveness of a vegetative cover over long-term periods should be questioned unless other protective schemes, such as armoring of the cover with the proper size material, are also included in the design.

5.1.2.5 Example Problem

An example problem in how to use the MUSLE is provided below.

Assumptions:

Site location:	Western Colorado
Site description:	Uncovered tailings pond
Pond size:	160 acres
Slope:	3%
Length:	2500 ft
Material:	42% sand greater than 0.10 mm; 58% fine sand and silt less than 0.10 mm; 5% clay less than 0.002 mm; 0% organics; (53% silt plus fine sand less than 0.1 mm); Consistency - fine granular; Permeability - slow to moderate.

The following factors have been determined for use in Equation 5.1.

R = 20 from Table 5.1

K = 0.50 from Figure 5.1

LS = 0.747 from Equation 5.2 and Table 5.2

VM = 1.0 (average from Table 5.3 based on an undisturbed surface)

Using Equation 5.1, the annual soil loss (A) from the tailings pond due to sheet erosion caused by flowing water is computed to be 7.47 tons/acre/year, or 1195 tons/year from the facility. Therefore, the cover is estimated to erode at a rate of 0.003 ft per year, or 0.3 ft/century.

5.2 SUMMARY AND FUTURE STUDIES

The main application of the soil loss equation approach in the evaluation of cover integrity is to determine whether it is possible for sheet erosion to penetrate the tailings cover, thereby exposing bare tailings and constituting a failure of the cover. The followup study will concentrate

Table 5.3. Typical VM Factor Values Reported in the Literature.[a]

Condition	VM Factor
1. Bare soil conditions	
freshly disked to 6-8 inches	1.00
after one rain	0.89
loose to 12 inches smooth	0.90
loose to 12 inches rough	0.80
compacted bulldozer scraped up and down	1.30
same except root raked	1.20
compacted bulldozer scraped across slope	1.20
same except root raked across	0.90
rough irregular tracked all directions	0.90
seed and fertilizer, fresh	0.64
same after six months	0.54
seed, fertilizer, and 12 months chemical	0.38
not tilled algae crusted	0.01
tilled algae crusted	0.02
compacted fill	1.24 - 1.71
undisturbed except scraped	0.66 - 1.30
scarified only	0.76 - 1.31
sawdust 2 inches deep, disked in	0.61
2. Asphalt emulsion on bare soil	
1250 gallons/acre	0.02
1210 gallons/acre	0.01 - 0.019
605 gallons/acre	0.14 - 0.57
302 gallons/acre	0.28 - 0.60
151 gallons/acre	0.65 - 0.70
3. Dust binder	
605 gallons/acre	1.05
1210 gallons/acre	0.29 - 0.78
4. Other chemicals	
1000 lb. fiber Glass Roving with 60-150 gallons asphalt emulsion/acre	0.01 - 0.05
Aquatain	0.68
Aerospray 70, 10 percent cover	0.94
Curasol AE	0.30 - 0.48
Petroset SB	0.40 - 0.66
PVA	0.71 - 0.90
Terra-Tack	0.66
Wood fiber slurry, 1000 lb/acre fresh[b]	0.05
Wood fiber slurry, 1400 lb/acre fresh[b]	0.01 - 0.02
Wood fiber slurry, 3500 lb/acre fresh[b]	0.10
5. Seedings	
temporary, 0 to 60 days	0.40
temporary, after 60 days	0.05
permanent, 0 to 60 days	0.40
permanent, 2 to 12 months	0.05
permanent, after 12 months	0.01
6. Brush	
7. Excelsior blanket with plastic net	0.04 - 0.10

[a]Note the variation in values of VM factors reported by different researchers for the same measures. References containing details of research which produced these VM values are included in NCHRP Project 16-3 report, "Erosion Control During Highway Construction, Vol. III. Bibliography of Water and Wind Erosion Control References," Transportation Research Board, 2101 Constitution Avenue, Washington, DC 20418.

[b]This material is commonly referred to as hydromulch.